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TRANSACTIONS.

Paper No. 1062.

THE BRACING OF TRENCHES AND TUNNELS, WITH PRACTICAL FORMULAS FOR EARTH PRESSURES.*

By J. C. Meem, M. Am. Soc. C. E.

With Discussion by Messrs. Horace J. Howe, C. W. Birch-Nord, Lazarus White, E. G. Haines, F. T. Llewellyn, T. Kennard Thomson, Ernst F. Jonson, Francis L. Pruyn, R. A. Shailer, H. P. Moran, Eugene W. Stern, G. L. Christian, V. H. Hewes, E. P. Goodrich, J. F. O'Rourke, O. F. Nichols, Walter H. Gahagan, Milo S. Ketchum, Charles F. Marsh, F. L. Cranford, Jason Paige, L. R. Gifford, and J. C. Meem.

In this paper the sheathing and bracing of trenches and tunnels in dry and water-bearing materials, will be treated under the general subject of bracing. In order that there may be no misunderstanding, the term "sheeting" will be used for that class of sheathing which is set in or driven coincidentally with the excavation. That class of sheathing which is driven ahead of the excavation, or beyond its final limits, will be referred to as "sheet-piling."

Ordinarily, sheeting is set in or driven by hand-mauls, whereas sheet-piling is driven by pile-drivers. In order to make the descriptions as clear as possible, reference is made to Figs. 1 to 4, which show in a general way the different types of sheeting and bracing.

Fig. 1 shows a general type of open trench sheeted with two sets of, or "double," sheeting.

* Presented at the meeting of October 16th, 1907.

Fig. 2 shows a detail of the principal features of the sheeting (ordinarily 2 or 3-in. spruce or Virginia pine), the ranger with the engaging brace, and the cleats and lugs for holding them in position.

Fig. 3 shows a general type of excavation in a coffer-dam, sheeted with 12 by 12-in. tongued and grooved sheet-piling, driven between guide-wales, and penetrating beyond the limits of the excavation, but not sufficiently far to do away with the necessity for bracing. These guide-wales are usually bolted to ordinary piles driven ahead of the sheet-piling and removed as it approaches them.

Fig 4 shows the cross-section of a general type of hand-driven tunnel or drift. The cap, legs, and sill are clearly defined, with a spreader under the cap, the rear ends of the front lagging resting on the cap, and the front ends of the rear lagging superimposed above everything, with the waling pieces and fillers and wedges between. The same terms hold good for the sides. The fillers shown at the right of the sketch just over the cap are put in temporarily to hold the waling piece in position until the lagging or poling boards can be driven.

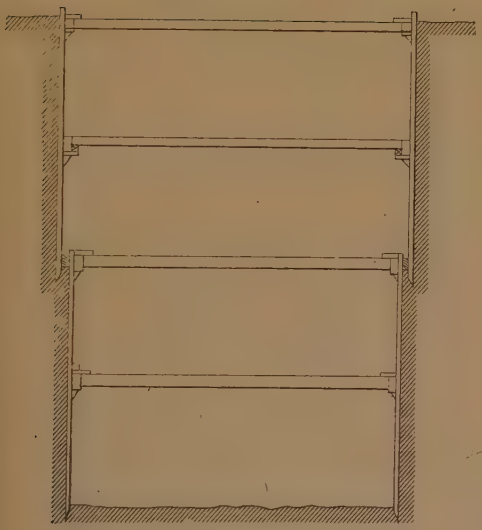
Referring now to the general question of earth pressure, in connection with its action on sheeting and bracing, the writer believes that this subject has never been developed so as to reconcile the theoretical with the practical conditions. It will be his endeavor to develop a practical basis in connection with which it will be possible at all times to effect an approximate reconciliation between the actual conditions of stability of earth and the theoretical formulas or resultants arising therefrom.

In all his experience, the writer has used the diagram, Fig. 5, for calculations of earth pressure, whether applied to retaining walls or to sheeting and bracing.

If $B\ C$ be the line of the sheeting, and $D\ C$ the natural slope of the earth, b being the angle of repose, then the mass of earth causing pressure against the line, $B\ C$, is contained within the triangle, $D\ B\ C$.

The weight of the earth in this triangle rests on $D\ C$, and its thrust is transmitted to $B\ C$, not through the toe of each layer at the foot of its slope line, but by the arching effect of this earth between the lines, $B\ C$ and $D\ C$.

For purposes of calculation, it is probably not far from correct to assume that a line, $A\ C$, bisecting this angle, $D\ C\ B$, measures

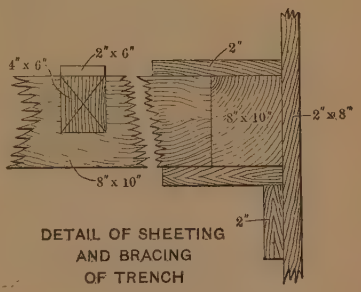


GENERAL TYPE OF OPEN TRENCH
WITH TWO SETS OF SHEETING

Scale of Feet

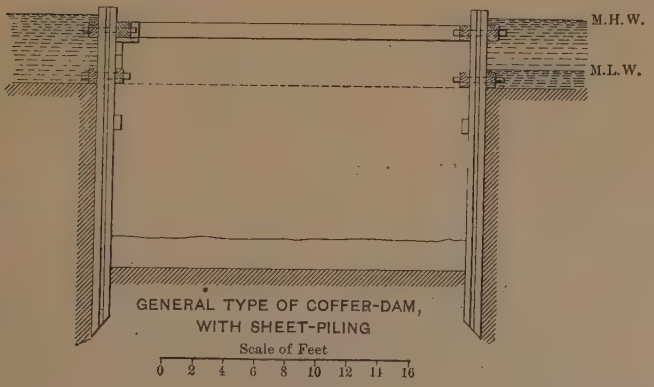
0 2 4 6 8 10 12 14 16

FIG. 1.



DETAIL OF SHEETING
AND BRACING
OF TRENCH

FIG. 2.



GENERAL TYPE OF COFFER-DAM,
WITH SHEET-PILING

Scale of Feet

0 2 4 6 8 10 12 14 16

FIG. 3

with *B C* an area equivalent to the weight transmitted as thrust against this line, *B C*. Also, it is true that the center of pressure against *B C* is where a perpendicular from the center of gravity of the triangle, *A C B*, meets this line.

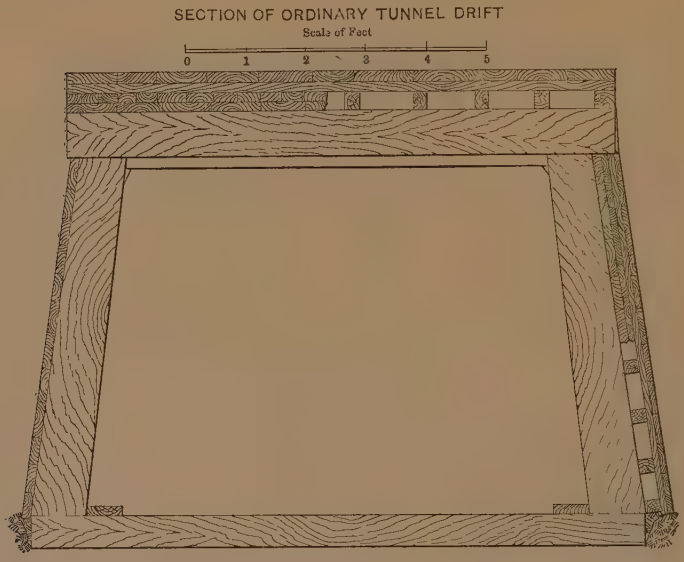


FIG. 4.

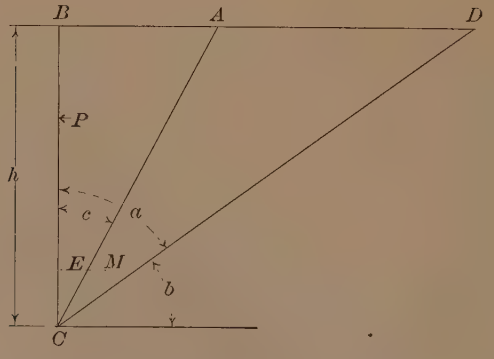


FIG. 5.

The writer is fully aware that in making this assumption he is going contrary to the general theory, which assumes that earth pressure acts along the line of rupture and parallel with that line, and is therefore greatest at the toe, but he wishes to state that this

theory is not borne out in actual practice, and that all closely-sheeted, well-braced trenches invariably show a heavier pressure at the top than at the bottom. Any attempt to assume a theoretical condition which is contrary to this fact must be of little value to engineers in making practical calculations. The writer is aware that many retaining walls have been built from designs based on this theory; and he may state parenthetically that it makes little difference, practically, which form of reasoning is applied to retaining walls. It does, however, make a vast difference which form of reasoning is applied to a braced trench, or to a concrete wall reinforced horizontally; and, while the writer does not wish to contradict this theory, if based entirely on a theoretical condition of frictionless material, he does wish to advise against its use in ordinary practice. In other words, if it be assumed that *D C* is a solid plane, and that the triangle, *D C B*, is filled entirely with particles which are absolutely without friction, and therefore the weight of one upon the other transmits accumulatively the weight of all entirely and directly to the bottom, then the truth of the theory noted above cannot be controverted. The writer is convinced, however, that such a condition does not exist, and that earth pressures and aqueous pressures are not similar; for, in dealing with ordinary materials, it is impossible to proceed with any practical calculations without taking into account the frictional resistance of these materials and their arching effects, which render it virtually impossible to consider their action as in any way allied to hydrostatic action. For instance, the writer has repeatedly observed (where trenches have been sheeted from *B* to *E*, Fig. 5, and the excavation has been continued below *E* without sheeting), undercutting excavations which have been made (in loams, clays, or moist sands) back along the line, *E M*, by the use of light poling-boards driven in under the toe of the sheeting at *E*, without disturbing the stability of the mass above. He is satisfied that if this sheeting had been so poorly put in as to cause the stability of the mass to be disturbed, it would have manifested itself by the continual dropping of masses of material from above, rather than as a constant pressure, as would have been the case had the material been full of water or absolutely frictionless. And, if any small holes should be left between these poling-boards in absolutely dry

sand, it would "bleed" in through these orifices just as the sand runs into an hour-glass, until the hole had gone to the top of the excavation, but at no time would there be observed any continuous pressure which could be defined in any way as equal to the entire weight of the sand from the bottom to the top over the unsheeted area. It may be, and the writer has frequently observed also, that the lower part of a trench may be left unsheeted, as from E to C, and for a considerable distance longitudinally in clays and moist sands, without disturbing the stability of the face, E C, and yet more or less heavy pressure may be observed in the bracing above. This can only be explained on the theory of the arching effect of the material above, one buttress of the arch being B E, and the other D C. If this be correct, there can be no doubt of the action being wedge-like, with the center of pressure opposite the center of gravity. Any engineer who has had to do with excavations must be aware of the fact that pressures are frequently developed in the top braces and rangers while men are excavating with impunity beyond the limits of the sheeting at the bottom of the trench. It is possible, also, at any time to cut or remove the bottom sheeting (except in dry sand) for a considerable percentage of the vertical distance from the bottom, and for indefinite lengths, without interfering with the stability of the bank above, provided the sheeting is removed without jarring. Any practical man, however, will admit that it would be suicidal to remove any one of the braces near the top of the excavation, particularly after the ground had stood for any considerable time.

The practical application of the foregoing will now be shown, and the formula be demonstrated.

If b (in Fig. 5) = the angle of repose,

$a = 90° - b$, and

$c = \frac{a}{2}$,

h = height = B C,

w = weight of 1 cu. ft. of earth.

Then, the area of A C B $= h \times \frac{h \tan. c}{2}$, and the weight of the mass of earth causing pressure on B C $= \frac{w h^2 \tan. c}{2}$.

The resultant pressure of this mass would occur at two-thirds of the height, or at P, in Fig. 5.

In the case of a well-sheeted and braced bank, there would be no overturning moment, but there would be a thrust, represented by the general tendency of the triangle, *A B C*, to slide along the line, *A C*, and therefore move out and exert pressure in a horizontal direction.

To understand this more clearly, it might be well to assume (in Fig. 6) that the lines, *D C A* and *D C B*, are blocks of ice which are held in place along the line, *B C*, by their weight impinging on this line, and on the line, *A C*, by a rigid strut bearing against that line at *P*. If the pressure at *P* be slightly released, the whole triangle, *A B C*, will assume a new position, A_1 B_1 C_2. If, again, *A D C* be taken as a solid wedge bearing against a solid block, *D C*, and the wedge be forced down, then, in

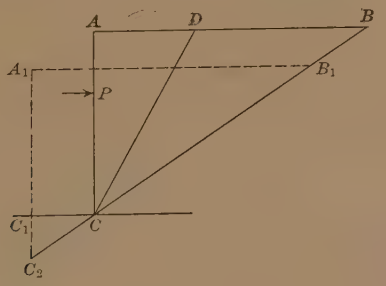


FIG. 6.

order that it may be resisted most effectually at any one point, this resistance should be placed at *P*. Or, in a word, the writer believes that the action of earth pressure in properly braced trenches is more closely allied to that of a coherent solid than to that of an aqueous or frictionless mass.

If, now, these pressures be applied to an ordinary sheeted and braced trench of an assumed depth of 40 ft., and if the weight of earth be assumed at 90 lb. per cu. ft., the pressures obtained at the braces shown in Fig. 7 will correspond with the figures in the rectangles opposite these braces. The maximum pressure on any portion of a trench sheeted in this manner would occur at the second brace, where the pressure for the assumed case would eventually be 8 640 lb. per ft. If these braces are spaced 10 ft. apart, it would be necessary to use a 12 by 12-in. yellow pine ranger to resist this

pressure (with a factor of safety of 3), and the third, fourth and other rangers would be of correspondingly smaller cross-sectional area. It is rarely found in practice, however, that it is necessary to use a ranger as heavy as that required by the theoretical conditions here assumed, for, if the trench be well-sheeted and braced, the horizontal arching effect of the material will come into play,

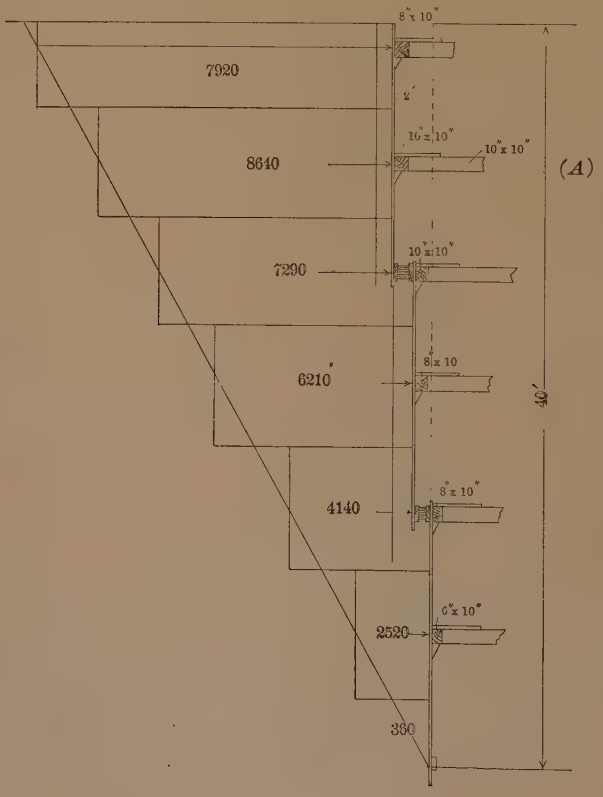


FIG. 7.

and will transmit itself from brace to brace and allow the use of a lighter ranger. It is also customary, within the limits of good practice, to use much lighter sheeting than that which would be theoretically required to maintain or resist this pressure, as the braces and rangers, if properly placed, take up the pressures and allow the arching effect previously noted to perform its work.

In Fig. 7 are noted the sizes and kinds of sheeting and bracing which would be used in ordinary practice in a trench 40 ft. in depth, and, while not recommended as theoretically strong enough, they give satisfactory results in ordinary ground where the trench does not have to stand too long, and always assuming that the bracing is properly put in originally. The top brace here shown is 8 by 10-in., the second and third are 10 by 10-in., the fourth and fifth are 8 by 10-in., and the bottom one may be a 6 by 10-in. These sizes apply to a good grade of Virginia or short-leaf yellow pine. Ordinarily, the toe of the sheeting is driven 1 ft. or more into the bottom material, which holds it in place without the necessity for bracing this toe. The braces sustaining the rangers (except in very narrow excavations) should be fully as large as the rangers themselves, as any tendency to distortion might be fatal to the stability of the banks; and, if the trench is very wide, longitudinal spreaders should be placed between the braces at sufficiently frequent intervals to prevent the possibility of distortion. These spreaders obviate the necessity of using heavier braces.

The writer believes that there is a limit of depth beyond which it is not possible to brace a trench against the pressures which would be developed, and he believes that this limit could be defined by a simple practical calculation, depending, of course, upon the nature of the soil through which the trench was dug. For example, if a trench be sunk 20 ft. and stopped, the pressure developed at the 15-ft. level will not be excessive, whereas if it be continued to 60 ft. the bracing will have to be heavily reinforced at the same 15-ft. level; and, if the trench be carried down to an indefinite depth, no bracing would eventually be able to withstand the pressures at this same point, owing to what may here be described in a homely way as the "topheaviness" of the bank. Anyone who has had to do with deep trenches or tunnels, however, must realize that an exposed face of earth is under no more pressure at the bottom of the deepest trench or tunnel than it is at the bottom of a shallow one.

Reference is here made to Fig. 1, Plate I, which shows the bottom of a lined tunnel (at a depth of some 70 ft.) which had been driven with an ordinary shield, and from the bottom of which several cast-iron plates had been removed. The places from which the plates were removed show an exposed face of moist sand, and

it is clearly evident that it is in as quiescent and undisturbed a state as though the exposure had been made only a few feet from the surface. The rod shown in the photograph is vertical.

The danger in sheeting a trench arises mainly from three causes:

a.—In driving the sheeting carelessly and allowing slips to occur behind it; or, in the case of clayey soils, not properly guarding against voids which may occur behind the sheeting. The natural tendency of earth eventually to fill these voids causes slips, which develop not only the full pressures theoretically provided for, but frequently, by reason of the shock incidental to the velocity of slip, cause increased pressures to impinge against the sheeting and bracing.

b.—In not fully tightening the braces by the use of wedges driven practically to refusal; and

c.—Because strata of quicksand may be uncovered in ordinary soil, thereby developing hydrostatic and unbalanced pressures on the sheeting and causing stresses not properly provided for.

The next consideration is that of bracing and sheeting and its relation to earth pressures in subaqueous or other soils so saturated as to be under hydrostatic pressure.

There appears to be no controversy as to the generally accepted theory and formula in connection herewith, and they will merely be discussed in order to preserve the continuity of this paper.

Fig. 8 shows graphically the theory of hydrostatic pressure, which assumes that it is cumulative vertically and constant in the horizontal planes, and that the center of pressure is always at a point one-third above the base. If, now, it be assumed that *A B* is sheet-piling which has been driven through water and into the ground at *C*, penetrating sufficiently far beyond *O* to cause it to act as a cantilever, then the moment tending to overturn this sheet-piling is as follows:

If h = the height (in Fig. 8),

w = the weight of 1 cu. ft. of water,

and, if the width of each pile be 1 ft., then

$$h \times \frac{h}{2} \times w = \frac{w h^2}{2} = \text{the weight bearing against each pile,}$$

and

$$\frac{w h^3}{6} = \text{the moment tending to overturn each pile.}$$

FIG. 1.—SAND IN REPOSE AT SIDE AND BOTTOM OF TUNNEL.

FIG. 2.—PIT, 6 FT. SQUARE, LINED WITH HORIZONTAL SHEETING.

If a brace be placed at the point, H_1, the piling becomes a beam with the load distributed over the whole area, the resultant being at P. The writer is not convinced that it is necessary to consider the full hydrostatic pressure on that portion of the pile represented in Fig. 8 by C O. He believes that the frictional resistance of the earth here prevents this full hydrostatic pressure from being developed, but he would not be willing to neglect this in making practical calculations, and therefore he believes it should always be taken at its full value. The same reasoning applies to subaqueous tunnels, or to any subaqueous structure in soil sufficiently permeable to admit water in reasonably large quantities.

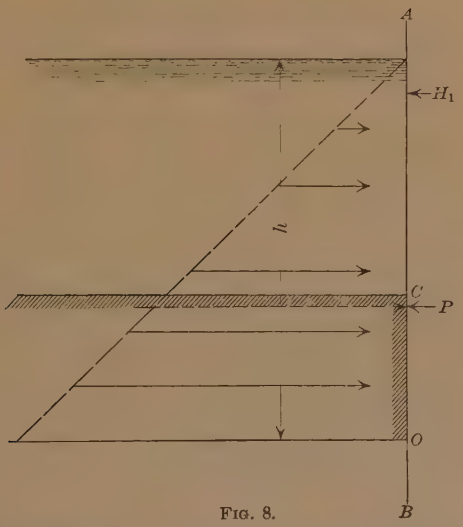


FIG. 8.

The next question to be taken up will be earth pressures on tunnels or subterranean structures, where it is necessary to consider the other side of the pressure areas noted in the development of the formulas for pressures in open trenches.

Referring to Fig. 9, if it be assumed that B D J I is a tunnel (the area of which will be taken as a square, in order to simplify the assumptions), and that H D or D F is the natural slope of the earth above this tunnel, then:

a = the angle of repose,

b = the complement of the angle of repose, and

$c = \frac{b}{2}$.

As a first assumption, it is unquestioned, of course, that all earth contained in the triangle, *B C D*, necessarily presses directly on the roof of the tunnel. And if the assumptions made at the beginning of this paper are true for open trenches, then it is also true that all the earth contained in the triangle, *A C D*, bears directly upon the line, *C D*, and therefore all this weight likewise is transmitted to the tunnel and all the pressure of the earth in the triangle, *A D L*, goes to the line, *L D*, arching itself somewhere below the triangle, *B A D*, in a curve approximating the line, *B S D*, so that, in a well-braced tunnel, the only pressure on

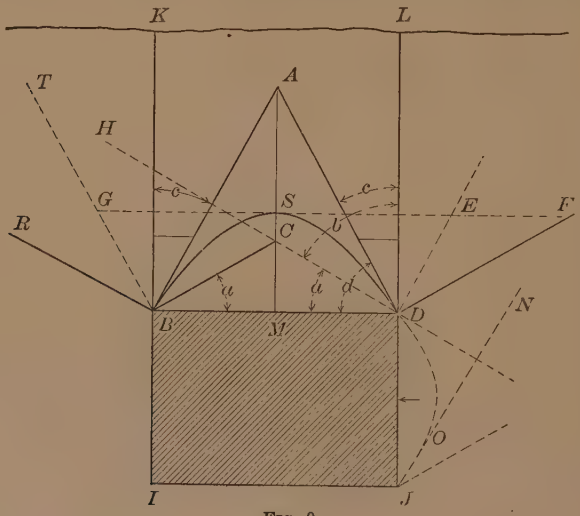


Fig. 9.

the roof would be that due to the weight of the material below this line. In order to be consistent and carry out the line of reasoning in the original assumption, however, provision must be made for the pressures of all the material in the triangle, *B A D*. No pressure beyond the lines, *B A* and *A D*, can be transmitted to the tunnel unless the ground contains water in such quantities as to make the pressure hydrostatic. Of course, if the tunnel is a subaqueous one, the pressure on the line, *B D*, for purposes of practical calculation, must be that due to the hydrostatic head of the water measured by its depth to the line, *B D*, and by the width of the tunnel opening.

Returning again to earth pressures free from excess of water, it is found, of course, that the larger the angle, *a*, the greater is the resultant pressure upon the tunnel. It is possible that some may argue from this that the greatest pressure will occur in clayey soil where the earth stands vertically. This supposition, however, is erroneous, because, any soil which will stand vertically and continue to stand vertically under any circumstances must be considered in the same way as solid rock, and cannot be properly classed as material for which bracing is necessary. As a matter of fact, however, clays are treacherous soils for tunneling, and frequently develop pressures by squeezing or sliding horizontally, for which it is difficult to provide.

If, now, it be assumed that the square, *B K L D*, is made up entirely of blocks of ice (or frictionless solids) having an angle of repose of 90°, the full weight of this material would have to be provided for in bracing across the tunnel roof, *B D*. If, on the other hand, a material, such as dry sand, be considered, in which the angle of repose is very flat, the arching effect of this material comes more greatly into play by reason of its tendency to slide along the angle of repose, and therefore, the condition of least pressure that can come upon a tunnel in dry ground is where the angle of repose of the superimposed material is least, always providing the material is held by close sheeting. In a word, then, the greater the angle of repose the greater the pressure to be provided for, and this leads to the conclusion that dry sand of a small angle of repose would prove the best material for tunneling if it were possible to sheet and brace the roof absolutely without disturbing its equilibrium. In practice, however, it is, of course, impracticable to drive a tunnel through sand of any kind without having some movement of material, and the dryer the sand, the more likely it is to run through any opening in the sheeting.

Erroneous ideas of tunnel pressures may be had from the fact that any bracing which admits the slightest settlement develops in consequence greater or less pressure, according to the degree of settlement, and the consequent movement, which is permitted to take place, but the writer is convinced, from long observation, that in reasonably careful work no pressures need be allowed for beyond the limits herein stated.

Here, again, it may be of interest to note that if a frictionless material could be imagined, resting in a trough made by the prolongation of the lines, *R B* and *D F*, measuring the angles of repose, allowance would undoubtedly have to be made for the full pressure of the weight of all the material contained within the prolongation of the lines of rupture, *B T* and *D E*. Assuming a case where a tunnel is so close to the surface that the arching effect is lost, for instance, if in Fig. 9 the surface of the ground be taken at *G E F*, it is probable that there would be obtained not only the full pressure of the ground directly above the opening, but an increase due to the lines of rupture, *B T* and *D E*. Therefore, in the writer's judgment, it is always wise to discontinue tunnel operations when the surface of the ground intersects, or nearly intersects, the perpendicular line, *A M;* and it is within the reasonable limits of good practice to tunnel when the surface of the ground is fairly well above the point, *A*. It may also be of practical interest to note that any longitudinal trenching should always be avoided if possible over the line of a tunnel while it is being excavated owing to the consequent destruction of the key to the arching effect of the ground.

As to the actual pressure:

Let l = the width of the tunnel;

$A\ M = h = \frac{l}{2} \times \tan.\ d.$

Then the area, $B\ A\ D = \frac{l\,h}{2}$;

and, assuming that $a = 34°$, then $c = 28°$ and $d = 62°$;
and the tangent of d = approximately 2;
and l, of course = h.

The area, $B\ A\ D$, therefore, becomes $\frac{l^2}{2}$, and $\frac{w\,l^2}{2}$ = the total weight per linear foot of tunnel, where w = the weight of earth per cubic foot.

As to side pressures, the pressure against the sheeting, *D J* (Fig. 9), continues along a line of rupture, *J N*, stopping at some indefinite point, which, for practical purposes, may be taken at *O*, and making the only actual pressure in closely-sheeted work approximately within a line measured by the curve, *D O J*. An excess allowance covering conditions of sheeting, ground, etc., in

which judgment is a large factor, should be used in all calculations relating to the question of side bracing, to give a proper factor of safety in connection with this somewhat indeterminate quantity.

As to the question of shaft bracing, Fig. 10 is a cross-section of a square shaft of an area sufficiently small to give the surrounding earth a tendency to arch itself horizontally, it being very probable that such a shaft of reasonably small dimensions and driven as here shown will develop pressures somewhat as indicated by the circumscribed circle. It appears to be reasonably fair to assume that a circular shaft of not too large diameter, driven in dry ground or moist sand, will arch itself so that very little pressure is exerted

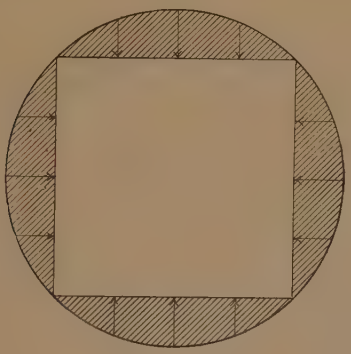


FIG. 10.

FIG. 11.

on the bracing in excess of that originally developed in making the sheeting and bracing tight. This excess pressure is small and indeterminate in actual practice, and may be measured by the intersection of the lines of the arching effect in a horizontal plane and the lines of rupture in a vertical plane, and somewhat as shown in the vertical section in Fig. 11. This pressure varies, of course, in direct proportion to the care with which the sheeting was originally placed. In view of this, it is true, both practically and theoretically, that a small shaft may be driven to any depth without developing any greater pressures below than are found near the surface. As soon, however, as the shaft becomes so large that the horizontal arching effect is destroyed, the action of the pressures becomes the same as the bracing in an open trench, and therefore

it would be impracticable to sink to any great depth a shaft the dimensions of which were such as to put it in the same category as trenches.

Attention is called to the photograph, Fig. 2, Plate I, which shows an underpinning pit sunk to a depth of about 18 ft. by the use of horizontal or well-diggers' sheeting. The bottom of this pit is at the level where ground-water has been struck, and is there sheeted with interlocking steel sheet-piling driven some 5 ft. into the ground to bring its toe well below the sub-grade of the adjoining excavation for which the underpinning is required. This pit can now be pumped out and excavated without danger of bringing in sand, and can then be filled with concrete to form a proper foundation for supporting underpinning timbers. The writer has personally supervised the work of sinking numbers of similar pits for different purposes, some to depths of 45 ft., without other bracing than the horizontal sheeting noted above, each set alternately bracing the other. It is also known that pits of this character, and not more than 5 ft. square, have been sunk by well-diggers to a much greater depth than those noted without using any bracing other than the sheeting of the character described above.

The writer believes that the practice of lining circular manholes with masonry walls the thickness of which increases with the depth is not consistent with good designing, and that a circular shaft may be safely designed with a masonry wall which has the same thickness at an indefinite depth as it has near the surface.

The remainder of this paper illustrates and discusses a few general types and methods of sheeting and bracing.

The ordinary sheeting, of course, is hand-driven vertically, as previously illustrated, but it is not always possible to drive sheeting, particularly in subway work, where much of the excavation has to be done under cover. In such cases, however, it is always possible to use cross, box, or horizontal sheeting, types of which have been illustrated in connection with the sinking of pits. This type of sheeting, of course, is more expensive to put in, but in some instances gives better results than its hand-driven prototype, as the work can be done more carefully and can be more closely watched to avoid the occurrence of voids behind it. It may be of passing interest to note that the writer on one occasion found a

void larger in size than the hand of an ordinary man behind sheeting which had stood for fifteen years in a clayey soil.* He does not desire to offer this as a reason, however, or even a good excuse, for leaving such voids.

The general type of bracing shown in Fig. 12 may be taken to illustrate the more usual methods in connection with openings as

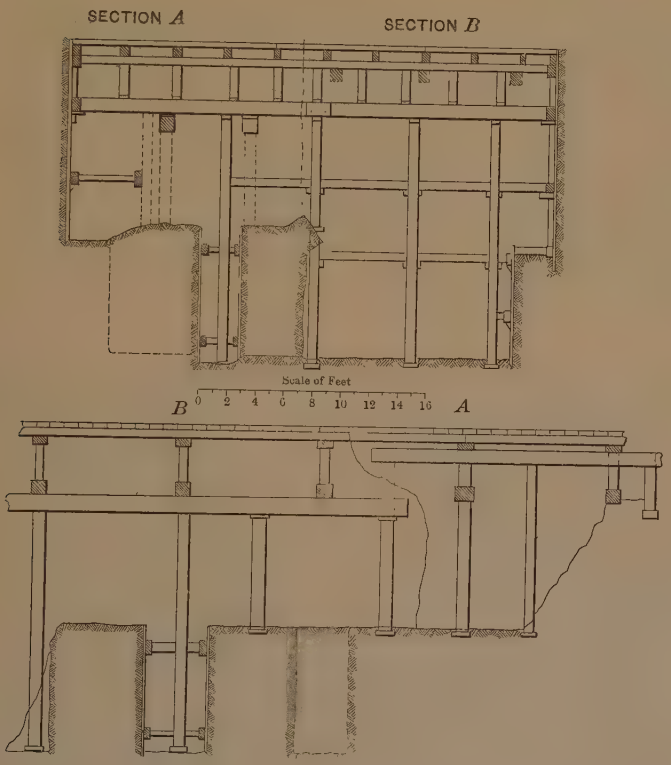


FIG. 12.

large as those required for the ordinary two-track subway. The general scheme in this is to dig the excavation in two or more levels, according to the depth, completing the excavation for a portion of one level before the other is begun. The decking is put in and

* Since preparing this paper the writer's attention has been called to a void recently found in excavating over a tunnel driven fifteen years before in sand. This void was large enough to admit one-half the body of a man.

carried on longitudinal sills, which, in turn, are braced from a lateral cap or caps, as shown, and where interference of pipes does not prevent. From this point the main cap is carried on temporary longitudinal I-beams or timbers of sufficiently large section and length to span from the solid ground ahead to the posting behind and carry one or more caps, according to the size and weight overhead. In carrying the excavation to its final level, after the first portion has been excavated, long posts, reaching from the main cap to the bottom, are put in in pits, as shown; or, if the longitudinal bars supporting the caps are sufficiently heavy and the superimposed weight is not too great, they may be put in in open excava-

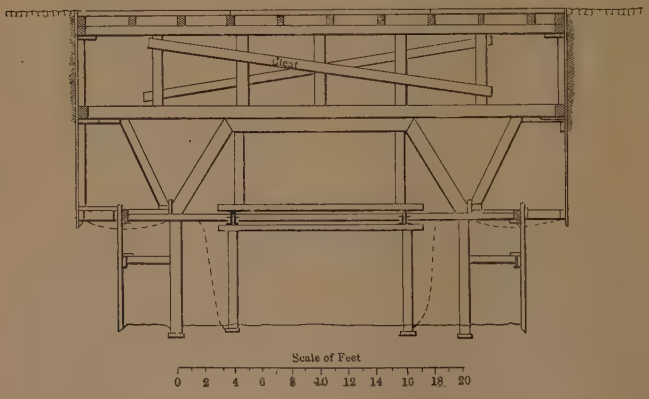


FIG. 13.

tion, as shown at the upper right portion of Fig. 12. The advantage of operating by this method is that the long posts, when once in place, give a vertical stability to the structure; but at the same time they are more or less cumbersome to handle, and not infrequently have to be cut up to be removed. Of course, the presence of pipes or other sub-surface or surface structures must change the form of bracing very materially, and the nature of the ground may also make considerable change in the method of operation.

In Fig. 13 is shown a type of bracing which has been used by Cranford and McNamee, in Brooklyn, in connection with their subway work. In this the decking where required is carried on longitudinal stringers, and where there are trolley tracks they are

FIG. 1.—TYPE OF BRACING IN FULTON ST. SUBWAY, BROOKLYN.

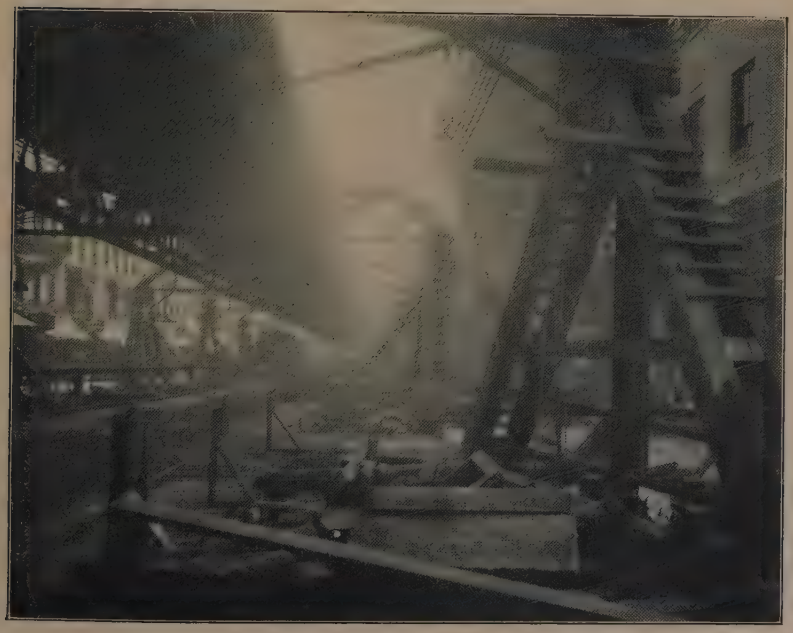


FIG. 2.—A-FRAMES, SUPPORTING ELEVATED RAILWAY, OVER SUBWAY EXCAVATION, BROOKLYN.

carried on longitudinal I-beams. As close as possible to the top of the trench, and below the general line of the pipes, needle-braces are run from ranger to ranger, and below this a cap is set. This needle and cap are carried on temporary posts and rakers until the heavy side rakers or inclined legs shown in Fig. 13 can be put in. The built-up, continuous, channel-beams, shown in cross-section beneath the main cap, are then run ahead, and the posting is carried up from that. The excavation is then carried down along the dotted line (and sheeted, if necessary), and the main posts are put in under the channel beams, the load being carried on the diagonals while this is being done. The excavation is then widened to its full limit.

The "bench" shown on each side of the excavation serves the double purpose of allowing for the construction of the side sewers and forming a break in the pressure area measuring the thrust of the bank. It is believed that, even where sewers are not required, it is a wise precaution to maintain some form of bench at or near this point.

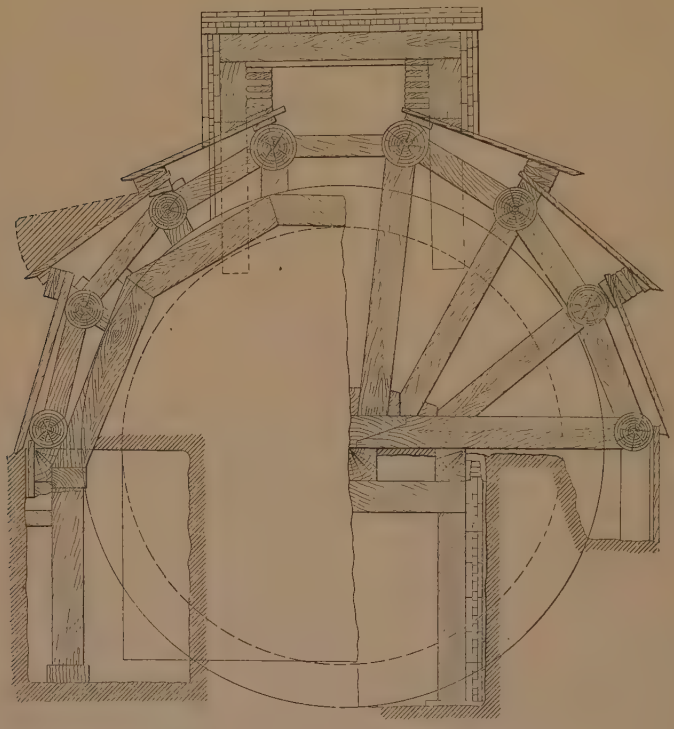
The advantages of this type of bracing are that it does away with the necessity of using long, heavy posts and temporary, long, heavy beams, as the longitudinal channel-beams make a tie which gives absolute stability to the shorter posting.

The interesting feature of Fig. 13, in connection with this paper, is that the heavy main brace, called the needle-brace, which runs from ranger to ranger, was calculated to do the heaviest work of the bracing, and was put by the contractors as nearly as possible at what they conceived to be the center of the heaviest pressure. The writer is thoroughly familiar with the work which this main brace has been called upon to do (in one instance it spans a continuous excavation for a six-track subway, and braces and carries the bracing for a double-track surface line and a double-track elevated railroad), and he can say positively, from his own knowledge, that at no time have the contractors ever seen fit to modify their first design of putting this main brace near the top rather than near the bottom of the trench.

Fig. 1, Plate II, is a view of a section of this type of bracing for a four-track subway in Fulton Street, Brooklyn, and Fig. 2, Plate II, shows the surface lines and bracing carrying the

columns of the elevated railroad, the supports of which are in turn braced directly by the timbering shown in Fig. 1, Plate II.

In tunnel bracing, the writer is more familiar with two general types of timber-braced tunnels. One of these is the crown-bar



Alternate Views:
Right View shows Central Bottom Drift.
Left View shows Lower Drifts on side to set in Leg for Wall-plate.

Scale of Feet
0 1 2 3 4 5 6 7 8 9 10

FIG. 14.

system, shown in Fig. 14. It is not necessary to do more than call attention to the general features of this type. In the ordinary crown-bar system it is customary to drive the bottom drift ahead of all other work and follow this closely with a top drift, as shown

in Fig. 14. The bars are placed in position in this top drift, the poling-boards are driven off approximately as shown, and the bracing is erected as the excavation proceeds. Where this type of bracing is used in rock tunnels it is, of course, unnecessary to drive a bottom drift, and the arch timber bracing shown on the left is usually put in underneath the bars as soon as the depth of excavation corresponding to the springing line of the structure has been reached. It is thus seen from Fig. 14 that, for the bar system alone, a circular tunnel of the size shown by the outer ring could be constructed, whereas with the arch timber bracing remaining in place, a tunnel no larger than that shown by the inner ring could be built.

The disadvantages of both these types of bracing are that so much space is required beyond the limits of the structure, and that it is impracticable to fill the voids over the masonry, even where dry packing of rock or brick is used; and it is virtually impossible to backfill them properly with sand or earth. Another disadvantage is the very prolific cause of settlement on account of the fact that the poling-boards do not finally rest in the plane at which they were started. This gradual lowering of the ends of these boards necessarily causes loosening of the material above and consequent settlement, with increased loading on the bracing, and while these items may not be of material importance in open country, they should be given the gravest consideration where this system is used under city streets.

Where arch timbers have been used, as in a few cases, in connection with the crown-bar system, and put in sufficiently high to clear the outer ring of the tunnel, it has been sometimes customary to run two bottom drifts ahead (as shown at the left of Fig. 14), setting in supports for the wall-plates before the upper excavation has been made. There are some instances on record in which the wall-plate has been directly underpinned after being put in place with a load on it. The writer has supervised the work of building tunnels of large diameter, and in one of the headings the arch timbers were used in connection with the crown bars and afterward removed; but in the remainder of the work with which he is familiar the arch timbers were used in connection with a special type of timbering shown in Fig. 15, all of which was eventually removed, leaving the tunnel sheeting bearing directly on the brick-

work. In this type the arch timbers were underpinned from the masonry which was put in in a centrally sheeted and braced trench, as shown, the entire masonry of the structure being built in three successive stages, of which the first is that noted in the bottom of the trench. In this method the outward thrust of the bottom legs of the arch, or segmental timbers, precludes almost entirely the possibility of any settlement while the sheeting for the middle trench is being driven. In dry ground there is practically no danger of any settlement to be caused from this method of under-

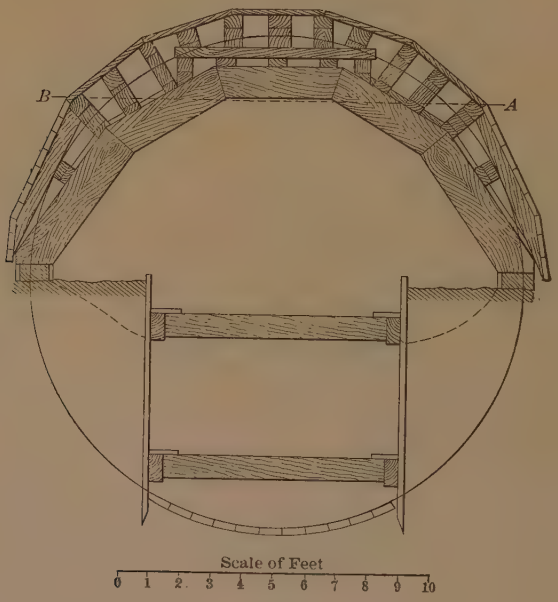


FIG. 15.

pinning, and the writer has supervised tunnels built safely by this method, where the ground-water level was normally 5 ft. above the sub-grade and had to be kept down by constant and heavy pumping through sand.

It may be of interest to state, in connection with the action of earth pressures, that it is almost always practicable to set in horizontal sheeting below the line, *A B*, in Fig. 15. In cases where the sand is moist and not too gravelly, very little difficulty is ex-

ARCH TIMBERS SUPPORTING RAISED TUNNEL ROOF.

perienced in making the excavation for the width of a plank, but where the ground contains gravel, boulders or sharp sand, it is necessary to give some little protection to the outer face of this material while the next plank is being put in place. This protection is usually secured by using thin strips, such as barrel-staves or splinters, backed by hay used in the form of thatch. These splinters or barrel-staves may be pulled ahead as each plank is set in place.

Plate III is a photograph of a section of tunnel braced by arch timbers, of five segments each. These timbers are 12 by 12-in. short-leaf pine, and are subjected to great stress in supporting a continuous section of the cast-iron roof (not shown in the photograph), which has been cut out and jacked up preparatory to the admission of a new lining. The writer believes that this form of timbering is the most substantial that can be introduced into tunneling.

The consideration of the questions contained in this paper has long been in the writer's mind, and he has at last decided to present them, for two reasons:

First, he believes that the subject of bracing and its relation to earth pressures is not properly provided for in the ordinary data and formulas, and he hopes, either in this paper or in the resultant discussion, that the question may be established beyond controversy, and on a basis which will be of practical value to engineers and contractors.

Second, he believes that insufficient consideration is given to the subject of bracing by engineers in general, it being ordinarily deemed sufficient to leave this question to more or less intelligent foremen; and, while it is not intended to impeach in any way the intelligence of any foreman, for whose opinion the writer has in many cases a very high regard, it is thought that the subject is one requiring more engineering consideration than is ordinarily given to it.

In conclusion, the writer wishes to thank George S. Rice, M. Am. Soc. C. E., of the Rapid Transit Railroad Commission; George H. Pegram, M. Am. Soc. C. E., of the Rapid Transit Subway Construction Company; and Mr. F. L. Cranford, of Cranford and McNamee, for the use of photographs, and for valuable suggestions.

DISCUSSION.

Mr. Howe. HORACE J. HOWE, M. AM. SOC. C. E. (by letter).—The result of a rest after disturbance is an important factor to be considered in earth pressures. This is true, whether it is a case of adhesion to a pile, or of adhesion to a row of sheet-piling. The effects of weather, water, frost, shocks, and vibrations must all be reckoned with.

In city work, the evils due to digging holes, or opening up ahead in order to show progress, are apt to be impressive where delay from various causes ensues, so that perhaps a job stands over winter or longer. Quick operation spares many a patient, and the thought persistently comes, with respect to attempts at formulating probabilities, that what might be accurate, allowable, and advisable for an energetic construction gang with a clear field and all obstacles, legal and official, removed, might be questionable when the above agencies have full play for a considerable time in a busy street.

The author sees fit to reverse the customary two-thirds height, of the authorities, and consequently increases the top relative to the bottom bracing, as shown in Fig. 7. This is a good thing to emphasize practically. One reason is that larger openings are thereby made; intermediates are not required; and buckets can have free swing when at full speed upward; but, owing to changes of cleavage of the earth, due to the above causes, the writer believes that the center of pressure will tend to travel downward, and the proportions indicated will not be preserved. The dimensions in Fig. 7 would be liberal for a pit of ordinary width, say 12 ft.

The writer had occasion once to watch a contractor who used 6 by 6-in. stuff throughout, to a depth of about 35 ft. About half way down, the bracing showed strain, and made this size about the limit, in his experience. Two lines of surface cars straddled the hole, and added substantially to the packing around the sheeting, as time went on.

The writer has seen long, straggly braces, propped up by verticals, where the author's sizes would not be adequate, and where diagonals to the ground were evidently needed.

Fig. 1, Plate I, gives a view at 70 ft. depth after the removal of plates in moist sand. Whether under air pressure, and for how long a time the exposure was made, is not stated. This quiescence indicates lack of pressure on the underside of a tunnel tube, and presumably on any similar foundation. Is it to be assumed, then, that nothing is gained by increased depth of foundation except greater compactness of soil?

The writer agrees that the use of the crown-bar system under a street leads to slumps, and trouble in general. Some years ago he observed an attempt made in a neighboring city by experienced contractors. The soil was reliable, but the work had to be taken out of

their hands, in the interest of safety to the public. The arch system would possibly have been more successful. Mr. Howe.

In conclusion, the writer is constrained to quote the opinion of that great formularizer, Rankine, on earth pressure. He says:*

"The properties of earth with respect to adhesion and friction are so variable, that the engineer should never trust to tables or to information obtained from books to guide him in designing earthworks, when he has it in his power to obtain the necessary data either by observation of existing earthworks in the same stratum, or by experiments."

C. W. Birch-Nord, Jun. Am. Soc. C. E. (by letter).—Mr. Meem's principal statement regarding the location of the resultant of the lateral forces on retaining walls seems to be somewhat different from what most engineers have been accustomed to use in their calculations. Mr. Birch-Nord.

Does it seem reasonable that a material with a certain angle of repose will act in a manner so entirely different from water that it will change the location of the resultant of the lateral forces from $\frac{1}{3}h$ from the base to $\frac{1}{3}h$ from the top?

Does it not seem reasonable that the maximum lateral pressure per unit should be greater near the base?

Mr. J. A. Jamieson, in his valuable paper, "Grain Pressures in Deep Bins,"† shows that the maximum lateral pressure per unit occurs near the bottom of the bins.

E. P. Goodrich, M. Am. Soc. C. E., in his paper, "Lateral Earth Pressures and Related Phenomena,"‡ gives results similar to those obtained by Mr. Jamieson, and he goes even so far as to prove that the resultant of the lateral forces is located between $0.38h$ and $0.40h$ from the base.

Mr. Meem illustrates his method of locating the resultant of lateral pressure by assuming two triangular-shaped pieces of ice lying on top of one another, and states that the resisting force, P, should be applied at $\frac{1}{3}h$ from the top, but, by making the experiment it will be found that the force, P, may be applied at any distance above the apex and still cause equilibrium.

There are many tangled, incomplete and different ideas regarding earth pressure and its action, and it is about time that something were done, in the line of extensive experiments, either by a Special Committee of the American Society of Civil Engineers or by some of the leading universities, in order to establish the facts in reference to this matter.

Lazarus White, Assoc. M. Am. Soc. C. E. (by letter).—During the writer's connection with the work of constructing the subway Mr. White.

* "Civil Engineering," page 317.

† *Transactions*, Can. Soc. C. E., Vol. XVII, p. 554; also *Engineering News*, March 10th, 1904, Vol. LI, p. 236.

‡ *Transactions*, Am. Soc. C. E., Vol. LIII, p. 272.

Mr. White. through Joralemon and Fulton Streets, in Brooklyn, he has frequently discussed with Mr. Meem his theory of earth pressures, and, although at first he held more nearly to the generally accepted view of earth pressures than that advocated by Mr. Meem, he has come to believe that his general views, when applied to the pressure exerted by materials without hydraulic properties, but with at least the cohesiveness of moist sand, are more nearly true than those accepted in the design of retaining walls since the time of Rankine. However, Mr. Meem says he would not apply his methods to the designing of retaining walls.

The photographs in the paper give ocular demonstration of the fact that earth pressures do not ordinarily increase with the depth, but that, on the contrary, in undisturbed material, they are almost zero at the bottom of the trench. It is with this point in view that Mr. Meem has solved so successfully numerous difficult problems in connection with the subway in Brooklyn. Along Fulton Street, in particular, a long line of buildings had to be underpinned, and the elevated railroad, trolley tracks and numerous pipes had to be supported. This work has advanced so far now, and Mr. Meem's methods have been used on so large a scale, that it can be fairly said that his ideas and methods of sheeting have been demonstrated experimentally on a very large scale.

It is true, as Mr. Meem says, that sheeting and bracing used in tunnels and open excavations are usually placed by foremen in a somewhat haphazard and happy-go-lucky way. In some cases the writer has seen enormous quantities of timber used to accomplish results obtained by Mr. Meem with a much smaller quantity, designed in advance and placed carefully by his methods. In the former case, bracing foremen have been allowed to place timbers wherever they thought them necessary, with the result that the trenches were encumbered with a mass of timber which seriously handicapped the placing of steel and masonry.

It will amply repay any engineer to make careful study of Mr. Meem's paper and the methods used by him. The most severe test of his methods was at the Joralemon Street Tunnel, where the double-track subway was excavated through coarse sand, and in close proximity to various flimsy buildings, without damaging them to any serious extent. In addition, the cast-iron tubes on the Brooklyn side of the Battery Tunnels were reconstructed under his direction. Such a reconstruction is considered by the writer as a feat which has never before been attempted, but Mr. Meem's modesty prevents him from giving it the importance it deserves. There is no field of engineering where poor and slipshod methods can accomplish the waste of so much time, money and even life as that in timbering and bracing soft ground. Mr. Meem has made considerable advancement in tunneling through soft ground, and in bracing deep trenches.

Mr. Haines.

E. G. Haines, Assoc. M. Am. Soc. C. E. (by letter).—This paper has proved of great interest to the writer, as he has been connected for several years with work requiring excavation in large quantities, in sheeted and open cuts, shafts, drifts, etc., and has had opportunity to observe the action of the materials under varying conditions; and, having contemplated writing a paper on the subject, he presents the substance of it now, as a discussion of Mr. Meem's paper.

Some time ago, after consulting all the available writings on the subject of earth pressure, and being unable to reconcile the results with his own observations, the writer became convinced that the common theory was erroneous, and that no practical formula could be developed by considering earth (unless either so dry or so saturated as to flow) either as a granular or as a fluid mass, but that it must be considered as a solid body, the characteristics of which it possesses.

The writer will endeavor to illustrate the result of a number of observations of "slips" or "caves," in both sheeted and open cuts, etc., and give his conclusions thereon, in the hope that they may be of some assistance in developing a practical formula for pressure.

In excavating open cuts without sheathing, with the sides nearly or quite vertical, slips frequently occur, the form of the fracture being very uniform (for the same material), and as shown by Fig. 16.

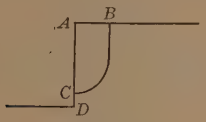


Fig. 16.

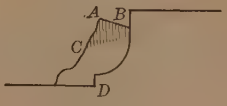


Fig. 17.

After the slide has subsided, or come to rest, it will have assumed a form such as shown by Fig. 17. The top surface, *A B*, will frequently be found inclined as shown, and quite a large volume of the material, *A B C*, quite undisturbed. The distance, *A B*, is usually about one-half *A D*, and, for any one material, keeps very closely constant.

It is very frequently noted that the toe of the fracture occurs at *C* (Fig. 16), some little height above *D*, and the writer was for some time unable to assign a reason. This will be considered later.

The plan of the slip is generally of the form shown by *A B C*, or *D B E*, in Fig. 18, while the vertical elevation of the face is as shown by Fig. 19. It may also be noted that in any material except dry sand, or in saturated ground, one may excavate to a certain depth without danger of slides; but, upon going deeper, slides are very apt to occur.

Taking next the case of a sheeted trench: It will usually be found that, for some little depth, the sheeting can be driven easily, and without evidence of much pressure. The pressure then becomes evident,

Mr. Haines. and a crack appears at the surface, as shown in Fig. 20, although the line of rupture may not appear at the bottom, even if the earth is exposed below the sheeting.

Again, with sheeting driven to some depth, and kept tightly keyed and filled, a cave will often occur at the bottom, as shown by Figs. 21 and 22, if the planks are not kept well footed, and this without showing any sign of a fracture of the whole bank, at the bottom or at the surface.

FIG. 18.

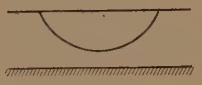


FIG. 19.

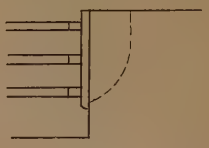


FIG. 20.

Again, the surface, after the first crack (in Fig. 20), may not crack farther back, even though the excavation be continued to considerably greater depth.

In the case of a tunnel heading or drift, it is frequently found that a small drift can be driven with perfect safety; but, upon at-

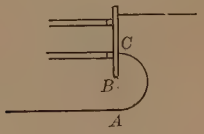


FIG. 21.

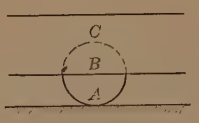


FIG. 22.

tempting to drive a larger section of the same form, in the same material, falls are of frequent occurrence, and appear about as in Fig. 23.

Taking also the case of shafts or pits, it will frequently be found that a circular pit can be sunk to some little depth, without caving, in material in which a square pit or trench will cave badly, as shown by Fig. 24.

FIG. 23.

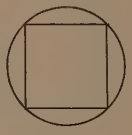


FIG. 24.

The three main classes of excavation have now been mentioned: a longitudinal trench, a horizontal drift, and a vertical pit; and, without carrying the matter further, the writer would state that in not one of these cases is there the slightest evidence of either a force similar to a hydrostatic pressure, or one similar to that required by

the theory for a finely-granulated mass. After the fall occurs, other conditions remaining unchanged, no further action takes place, often for extended periods of time. Mr. Haines.

The writer has examined many falls, under all the foregoing conditions. In quite a number of them he has had occasion to take measurements, and has been impressed with one feature common to all, namely, the shape of the fracture. In nearly every case, a cross-section of the fall, in one or any direction, is in the form of a curve approximating a segment of a circle. In other words, the fractured area forms a portion of the surface of a sphere.

This fact is important. It is totally inconsistent with the theory for fluid pressure, or that for a fine granular mass; but it is perfectly consistent if the earth is considered as an elastic solid body. This assumption may properly be made, as a mass of earth in its undisturbed state possesses the same properties of cohesion, weight, tensile, compressive and shearing strength, as does a mass of sandstone, but in much less amount.

The application of this the writer will now attempt to show.

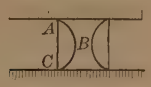


FIG. 25.

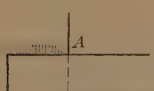


FIG. 26.

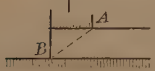


FIG. 27.

FIG. 28.

If a cube of stone, or other solid of granular structure, is submitted to compression between two parallel plane faces, it will fail, first by a shearing out of the sides, along the lines, *A B C*, Fig. 25, followed by a crushing down at *B*. If, instead of a cube, a slab is substituted, with the load applied as in Fig. 26, it will shear off from *A* to *B;* but, if the load be applied as in Fig. 27, and be free to move laterally, it will shear off from *A* to *B* as shown.

Now, this is exactly what takes place with a bank of earth, modified by the fact that such a bank must itself furnish the destructive agency, or the weight necessary to cause the shear.

It is known that the greatest area is contained within a given perimeter when it describes a circle; also, that the greatest volume is contained within a given superficial area when that area forms the surface of a sphere. Herein lies the secret of the spherical fracture.

Mr. Haines. To illustrate: take a bank of earth caved as shown in Fig. 28, in section and elevation.

If $D =$ the depth, $A\ C$, in feet;
$W =$ the weight per cubic foot;
$V =$ the volume, in cubic feet;
$S =$ the shearing strength per square foot;
and $A =$ the spherical surface, in square feet;
then, at the instant the cave occurs, $W\ V = A\ S$.

Now, for a sphere, $V = D^3 \frac{\pi}{6}$,

and $A = D^2 \pi$.

Therefore, the weight of the volume varies as the cube of the depth, while the area varies as the square of the depth; and, W being easily obtainable, if the value of S were known, the value of D could be found at once, and thus the depth which could be excavated without any caving of the bank. This would be of no practical value, however, as S would not remain a constant, but would change under weather and other influences, in some cases reducing to nearly zero; it is important, however, in the respect that it determines the section of the volume causing fracture, and, consequently, the pressure against any medium designed for its support.

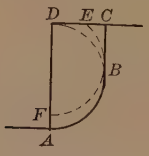


FIG. 29.

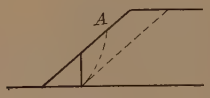


FIG. 30.

In case the slide is very long in proportion to its height, it is simply an extension of the spherical form, caused by continued falls, the form of the cross-section remaining unchanged. It still remains a semicircle, and explains why the crack at the top of a bank appears at a distance back equal to about one-half the depth, since the depth equals twice the radius of the curve of fracture.

In the case of an unsupported face, as in Fig. 29, the fracture does not complete the semicircle, $A\ B\ D$, but breaks out from B to C, as the distance is shorter, and, consequently, the line is one of less resistance.

The tendency to complete the curve, however, is shown clearly in many slips by the decided overhang at the top, as at E; and is shown perfectly in the case of a cave at the bottom side of a sheathed trench, as in Fig. 21; the distance, $B\ C$, seldom being much in excess of $A\ B$, if the sheathing has been kept keyed tight. In short, the earth does not "fall," but shears and slides out, revolving around the point, B,

Fig. 28; and, in an unsupported bank, the top surface will always be found on top after the slide has subsided, as was shown in Fig. 17. A cut of solid rock, if carried to sufficient depth, would act in the same manner. Mr. Haines.

A simple experiment will show the action which takes place. If, within a hemispherical cavity, there is placed a hemisphere of the same size, with its plane surface vertical (as shown again by Fig. 28), a certain pressure exerted below *B* will maintain it in its position; but, if the pressure be removed, the hemisphere will revolve until its plane surface becomes horizontal, as shown by the dotted line.

Mention has been made of the fact that the bank frequently breaks out at *F*, Fig. 29, instead of at *A*. This may be influenced by the change of the line of resistance, from *B D* to *B C*, though the writer does not now see how, but believes it to be due to the increase in the density of the material with increase of depth, and the tendency to follow the line of least resistance. It is not important, however, as the greatest area is contained within the lines, *A B C D*.

The author has presented a very simple theory, and a straight-line formula for pressure; but it appears to the writer to be defective, for the following reasons:

First.—It assumes, in effect, a material devoid of friction, which can hardly be assumed. Even in a material bearing large quantities of water, the adjoining excavation drains the excess of water, and the bank develops a large amount of frictional resistance.

Second.—The section of the bank causing pressure is not bounded by a straight line, but by a curve, as has been shown, part or all of which is a segment of a circle.

Third.—It is based on an angle of slope for the material, which is a factor not existing in a cut, but only in a fill, and then only when freshly made. The writer has frequently cut off the base of an embankment which had a well-defined angle of slope when deposited, and, although slips sometimes occurred, they did not extend to the top of the bank, but broke out as shown in Fig. 30, at *A*. The writer will admit that, given sufficient time, the face of an excavation would assume the same slope as would the material if placed in an embankment. It is not due to any cleavage plane in the material, however, but to the action of the elements. The surface is loosened by the rain, frost, and sun, and then water, wind, and the force of gravity move the loosened material, and pile it up at the base of the bank at its frictional angle of repose, Nature simply doing, after a long time, what Man, in forming embankments, does at once.

The writer recently examined a borrow-pit, on work with which he was connected fourteen years ago. The cut was about 25 ft. deep, the slope being left rough, anywhere between vertical and ¼ to 1. The material was principally sand, and the cuts of the railroad, in the

Mr. Haines. same material, were sloped to 1½ to 1. Numerous slips had occurred in this bank, all showing the curved line of fracture, but a large portion of the face still stood practically intact.

It appears to the writer that the pressure against sheathing is entirely due to arch action, and that in a consideration of the curved line of fracture should be found the basis of a pressure formula.

Let Fig. 31 illustrate a sheeted cut, caved at the bottom along *A B C*. The volume, *B C D E*, forms one-half of a perfect arch with semicircular intrados, and the crown thrust is taken up by the timbering and the compressive strength of the earth. This holds good for any position of the line of fracture, as shown by the dotted lines, and the arch will stand, unless the cave extend so close to the top, at *D*, that the pressure exceeds the compressive strength of the earth, or, the concentration of pressure at *B F* causes shear, or crushing of the area, *A B F*. The lines of compression must follow closely the lines of fracture, for, while the crushing value is fairly high, the value for shear is quite low, and any inclination of the compression to the fracture line would cause shear and continued falls of the material.

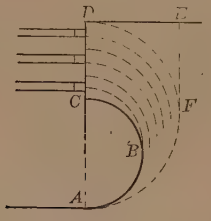


FIG. 31.

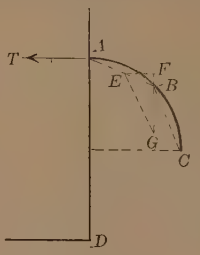


FIG. 32.

Considering the line of fracture to be the line of compression, the pressure at any point against the plane, *A D*, can easily be computed. This can be done graphically very quickly, as shown by Fig. 32. Let the line, *A B C*, be the center line of an arch strip of unit section, and let *L* equal its length. Also, let *W* equal the unit weight of the material. From *B* (the center of gravity of the line, *A C*) draw *B-G*, vertical, equal to *W L*, to some convenient scale. Completing the triangle of forces, *E-G* and *B-E* are equal to the resultant of the forces at *C* and *A*, respectively; and *E-F* is equal to the horizontal component of *B-E*, or, the horizontal pressure at $A = T$.

Assuming, for purposes of comparison, that $W = 1$, and solving for *T* at different heights, a series of coefficients is obtained, which, if multiplied by the actual value of *W* for any case, equals the pressure for that height. This is shown in Fig. 33, by the line, *A*, as well as the same factor for the author's method (assuming a natural slope of 1½ to 1), and the common theory for hydrostatic pressure, by the line, *C*.

The area, *D E F*, Fig. 31, however, has not yet been considered. This section, directly, cannot cause any pressure at *D*, because its area at that point is zero. Indirectly, however, it does exert a pressure along the whole height, *A D*, due to its dead weight, this pressure being zero at *A*, and a maximum at *D*. The total amount of arch thrust from this area can be found by taking *F* as a center of moments and computing the thrust at *D*. The writer believes this to be distributed, varying uniformly from *A* to *D*, and, converting its value to the form of a triangle, and adding to the line, *A*, in Fig. 33, the line, *D*, is obtained, giving the total coefficient of *W* and the unit pressure at any height. Mr. Haines.

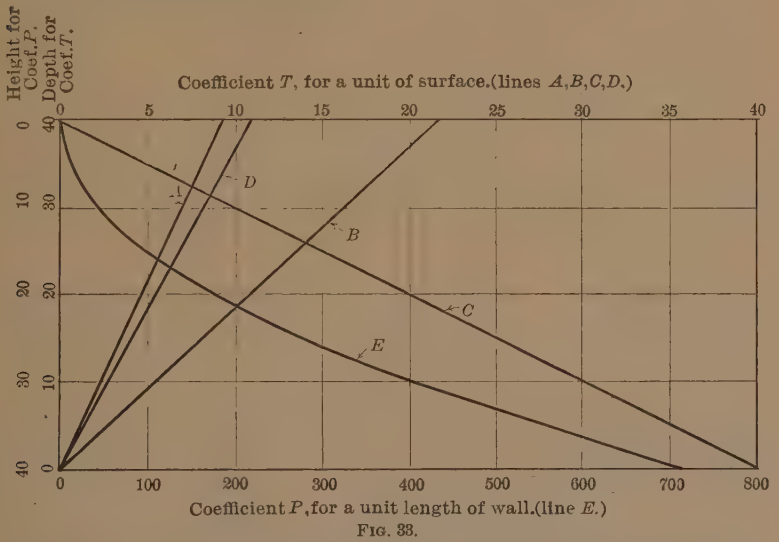


FIG. 33.

It will be noticed that this line gives pressures about half as great as those by the author's formula, and the writer believes that these are more nearly correct. He has never seen anything to indicate the extreme pressures shown by the author's formula, and Mr. Meem states that it is seldom necessary to use timbers as large as required by that formula.

Since the value of any theory depends upon its application to different conditions, the results, by the one proposed by the writer, will be compared with the conditions found in practice in widely different cases.

Taking first a shaft: If the material be fairly uniform, and the shaft can be excavated to a depth equal to its clear width without falls, there is little danger of falls at greater depth, and the reason is not hard to find. Let *A B C D*, Fig. 34, be such a shaft. If a

Mr. Haines. hemisphere, described on any side, be not heavy enough to cause shear over its surface, there is little danger of a larger fall (as shown by the dotted line), for it would then also have to crush a ring of the material, as at *D E*. The pressure likely to be found, then, at any depth, is due to local falls, and is equal to the arch thrust, shown on Fig. 33, for a height equal to the width of the shaft.

Fig. 34 also shows why a circular shaft is the safest form, as the hemisphere of dangerous weight has already been removed on each side, and the earth can fail only by the crushing of the material.

The writer, a few years ago, watched with considerable interest the excavation, in moist sand, of a well, about 8 ft. in diameter, without the use of sheathing. After the excavation had proceeded a few feet a wooden templet was placed, and an 8-in. brick lining was built to the surface. From that time forward the excavation proceeded by cutting under the templet, and the bricklaying was continued at the top, until, when last seen by the writer, a depth of 55 ft. had been reached. There was no evidence of crowding, or pressure, and the ring sunk readily, although the excavated material took a slope of about 1½ to 1. The foreman stated that he had often used this method for much greater depths.

Taking also the case of tunnel drifts in ordinary material, as shown by Fig. 35: If the sides, only, were of solid rock, a fall might be expected, as shown by the semicircle, *A B C*. Also, if the roof, only, were of rock, a cave might be expected, as shown by *A D E*. In abandoned mine workings, where falls are of frequent occurrence, the writer has found these conditions fulfilled, the earth seldom breaking back of the supporting rock, at *A* and *C*, and the height at *B* (unless the spherical surface be broken by boulders) not exceeding one-half *A C*, even though considerable water be flowing. In the case of a drift in clear earth, however, the timbering cannot be kept in perfect contact with the earth, and after the portions, *A B C A* and *A D E A*, have moved, there remains the section, *A B D A*, without support, except its cohesive strength, and it falls.

Thus it is seen that in firm ground the section causing pressure on a drift is described, from *F* on the axis of the drift, by a semicircle, with a radius equal to one-half the sum of the height and width, and completed by two quadrants, described from *G* and *H*, with a radius equal to one-half the height. Now, as to the actual pressures: The pressure against the cap, *B C*, Fig. 36, is due to the dead weight of the volume, *A B C D*, and varies as the ordinates between the lines, *A D* and *B C*.

The pressure against the side, *D E*, however, does not vary as the ordinates to the curve, but is due to the arching action of the material, as shown by the line, *A*, Fig. 33; and it is probable that most of the total pressure for the height, *D E*, is concentrated against the height, *C E*, as shown.

That the pressure at *C* is far in excess of that at *E* is shown clearly in practice by the fact that it is always necessary to strut between the top of the legs of the bents (unless they be mitered, or entered into the caps), while at the bottom they rest simply on a mud-sill. Mr. Haines.

Considering next the case of open cuts: The author has mentioned the case where it may be necessary to undercut the side of a sheeted trench, as shown by Fig. 37, and the writer recently saw such a case, the work being done by the author with perfect success. After the cut had been carried to about 20 ft. in depth, it was found necessary to excavate a gallery, about 200 ft. long, 5 ft. high and 7 or 8 ft. wide, along the side of the cut at the bottom. This was done by using poling boards from *A* to *B*, supporting them on wall-plates and posts, and by short vertical sheeting from *B* to *C*. This gallery was under a street carrying a double-track trolley line and heavy wagon traffic, and tremors from nearby elevated railroad structures could be plainly felt. The material was compact gravelly loam, with some small boulders. There was absolutely no evidence of any pressure against *B C*, the earth being exposed sometimes for several feet, and the plank

Fig. 34.

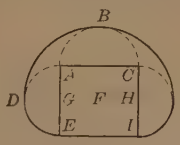


Fig. 35.

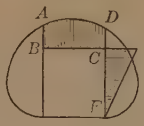


Fig. 36.

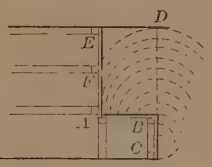


Fig. 37.

placed by hand. There was some evidence of load on the poling boards from *A* to *B*, but nothing like the full weight of the material above them. There was evidence of considerable load on the posts at *A*, however, and also a considerable thrust, laterally, on the timbers, *A*, *E*, and *F*, as shown by the fact that points given on the timbers, for use in the construction, moved from 1 to 2 in. This is exactly what might be expected from the arching of the material, as, the axis of the arch being on the line, *C D*, there is both a horizontal and vertical component to the thrust against the sheeting, as shown by the curved lines of thrust.

Thus far, the writer has confined his examples to what may be termed stable material. He will now go further, and say that it applies equally well to any material.

Taking first the case of dry sand: In the work of the Atlantic Avenue Improvement, in Brooklyn, of which the writer was Resident Engineer of a division, it was necessary to excavate a trench about 1½ miles long, 35 ft. wide, and about 25 ft. deep. On each side of the cut was a line of railroad track, carrying locomotives and heavy traffic, the center of the tracks being in some cases only 6 ft. from the sheet-

Mr. Haines. ing. On one side of the cut, 20 in. from the sheeting, there was also a 48-in. water main under pressure.

The contractor, wishing to withdraw as much of the sheeting as possible, placed it in two lengths. The top part was composed of 18 or 20-ft. planks, and the lower part was pieced out with 1¼-in. planks, set about 1 ft. in the ground at the bottom, and held at the top (from 5 to 7 ft. above) by cleats nailed to the under side of the bottom rangers. The material varied greatly, but in sections it consisted of sand so clean and fine that it filtered through the cracks in the sheeting and piled up at the bottom. A worse set of conditions could hardly be conceived, yet none of these light boards failed, nor did they show signs of great pressure, although 6 by 10-in. rangers cracked, near the top of the cut, in a number of cases. By referring to Fig. 38, the reason will be made clear. If a vessel be filled with fine sand, and an opening be made in the side, as shown at *A*, the sand will not flow out unless the height of the opening exceeds two-thirds of the thickness of the wall; but, if a larger opening be made, as at *B*, the sand will flow and pile up until its angle of slope reaches the top of the opening, and no more will then flow out. Within the vessel, the action still goes on; the sand falls from the bottom of the arch, and piles up at the bottom, until it finally breaks out at the top. This is purely an arching action, the falling material serving to support the base of the arch and building up a new arch as it is deposited, the lines of pressure being more clearly shown by Fig. 39.

The writer traced the line of first crack, on both sides of the work just mentioned, in the block paving of the street, for nearly the entire length of the work, and the ratio of width to depth appeared to be entirely independent of the character of the material; and, although the railroad track was sometimes within and sometimes without the fractured area, the first crack appeared very closely at a distance back equal to one-half the depth of the cut.

Turning now to saturated material: In the construction of a section of subway, in New York City, a trench was excavated about 45 ft. wide and 35 ft. deep, through ground which at some time had been filled into tidal water. The top surface of the ground was 7 ft. above mean high water, and the water appeared at that depth in the excavation, and constant pumping was necessary. The excavated material was so soft that it would slop out of the buckets, and the laborers had to work in rubber boots. The sheet-piling was driven by a pile driver, and the bottom 5 to 8 ft. of the cut was in rock. After the excavation was complete, a crack, about 2 in. wide, appeared at a distance back somewhat less than half the depth to the rock. It became evident that the timbering was under some stress near the top, but it would have held safely had it not been for an unfortunate accident which destroyed the timbering, and the cut caved in, on one

side for about 60 ft. in length, and somewhat less on the other. The material from these slips assumed such a flat slope that the two sides met in the bottom of the cut, and piled up as shown by Fig. 40, leaving exposed a height of about 20 ft. of the cave on one side. The top overhung the face, 8 ft. below, by nearly 3 ft., notwithstanding the fact that the base of a large pile of rock was within 6 ft. of the edge, and part of the footing for a stiff-leg derrick was actually beyond the exposed face below. It was several days before all the timbers could be replaced, and the slide refilled, yet it stood in this condition. The writer had a number of measurements taken, and, all things considered, it was the finest example of the spherical fracture he has ever seen. Mr. Haines.

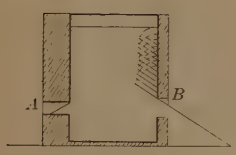


FIG. 38.

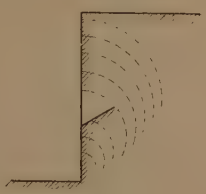


FIG. 39.

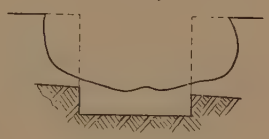


FIG. 40.

A mass of saturated earth closely resembles a mass of wet concrete, and Plate IV is a photograph, showing a cave in the latter material, due to insufficient bracing of the forms. The thickness of the wall was 3 ft., and the cave extended 4 ft. deep, the top being disturbed for about 2 ft. back from the form. The disturbed portion was all within the line drawn on the photograph, and the vertical face of the fracture is clearly shown at *A*. The portion marked *B* slightly overhung the fracture below. The concrete was placed quite wet, and none of it had been placed more than an hour, so that it could hardly be said to have set.

Taking, then, the design of a coffer-dam, with sheet-piling driven to some depth, as at *C*, Fig. 41 (*a*): As the water is pumped down, there is a uniformly increasing pressure against the sheet-piling, as shown by Fig. 41 (*b*). Below the level of the point, *B*, the material may be in either of two conditions, which may be termed water filled with earth, or, earth filled with water. In the first case, the material is what is called mud; it is not a fluid, nor does it act as one. It is insoluble mineral matter, and its presence in the fluid does not increase the density of the fluid, or its hydrostatic pressure, any more than it would if the grains were widely separated, and at points away from the sheeting. The water forms a perfect film against the sheeting, the same as it would in a box filled with billiard balls; and the pressure diagram for the water extends to the bottom at *C*, as shown by the triangle, *E F G*. The volume of earth, *B C D*, also exerts a

Mr. Haines. pressure against the sheeting, in the same manner as dry sand. The water leaking through the sheet-piling, or under the bottom, however, carries with it some of the earth, and tends to destroy the arch; and, unless more material be added, to allow it to rebuild itself, and the sheet-piling be kept well footed below the limits of the excavation, the arch will be entirely destroyed, and the unbalanced pressure from the other side will move the dam and cause its destruction. Thus there is, in this case, the arch pressure for the height, *B C*, added to the hydrostatic pressure, as shown by *G H I*. The amount of the pressure, however, is reduced by the buoyant effect of the water, and the value of *W* to be used with the coefficient is the difference between the weight of the water and the earth.

The second condition mentioned—that of earth filled with water—covers such material as shale, some clays, compact earth, and gravel. This material, while it passes water freely, somewhat resembles a series of screens, all made of wire of the same size, but of different mesh, the finer mesh being at the top, and increasing in size downward. This applies to a large number of materials, the finer and more compact material having been deposited at the top, and the writer has often produced this effect artificially by a covering of clay, a canvas mat, straw and manure, chaff, etc.

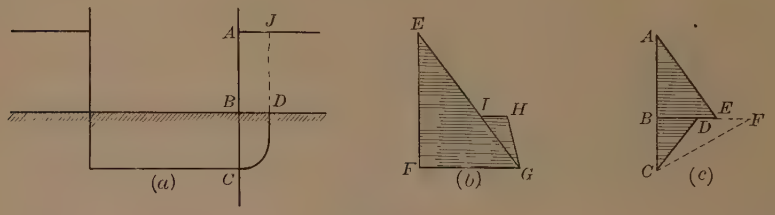
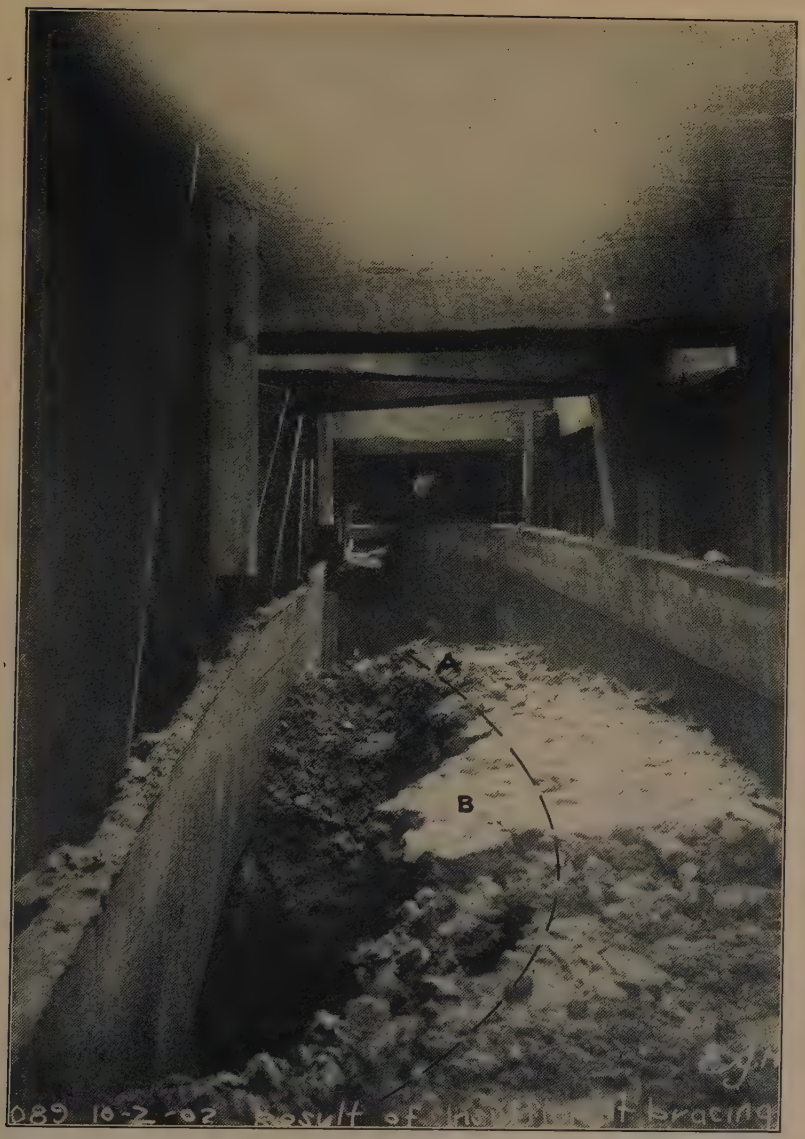


Fig. 41.

In this case, the pumping tends to remove the water faster than it can enter at the surface of the earth, and prevents any hydrostatic pressure below *B*. The water-pressure diagram is shown by *A B E*, in Fig. 41 (*c*). Below the point, *B*, there is the arch pressure due to the earth, but it is also carrying the weight of the column of water, shown by *A B D J* in Fig. 41 (*a*), and, in determining the factor, *W*, to use with the coefficient, it must be included in the weight of the earth, *B C D*, in Fig. 41 (*a*). The pressure diagram, then, is completed by the triangle, *B C D*, and, if the depth of the water be very great, the pressure below *B*, for a distance, may even exceed that due to the hydrostatic pressure, as shown by the dotted line, *C F*.

In subaqueous tunnel work, with a shield, and under air pressure, there obtains, probably, the worst set of conditions. Let Fig. 42 rep-



CAVE IN MASS OF WET CONCRETE, DUE TO INSUFFICIENT BRACING.

resent such a case. Here, there are peculiar conditions. If the air pressure is made equal to the head of water at the top of the tube, water will enter below that point, while, if the pressure be made great enough to keep out the water at the bottom, the air will escape rapidly above that point. Either of these things tends to destroy the arching effect of the earth. A common practice is to keep the pressure about equal to the head of water at the center of the tube, and allow some water to enter. This causes softening of the material, with consequent settlement, and the whole mass probably settles down along the lines, *A B C*. The excess of air pressure at the top causes erosion of the material, and "blow-outs" occur, followed by a reduction of air pressure. This reduction of pressure causes a rush of material toward the tube, and partially restores the arch just destroyed. That the disturbance is purely local is shown by the fact that a relatively small amount of clay, dumped in the water, is sufficient to close the break, and allow work to proceed, which would not be the case if one considered the angle of slope at which the material would stand.

Mr. Haines.

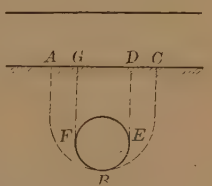


FIG. 42.

In the air chamber, the shield being a perfect arch, and also air-tight and water-tight, there is an arch of earth over the shield which is not subject to disturbance, and it probably carries most of its own weight. Were this not the case—the material at the bottom being much softened by the incoming water—the weight at the top would cause a dip to the shield which it would not be possible to control. Back of the shield, however, the rings as placed are not tight, and the air escaping at the top and the water entering at the bottom cause a continuous disturbance of the earth, and the rings probably carry a large portion of the weight, the water being taken care of by the air pressure. Back of the air locks, where the air pressure has been removed, the action of both water and earth is downward; and, as the joints are made tight, the earth becomes a perfect arch, supporting most, or all, of its own weight, and the rings are subjected simply to the hydrostatic pressure of the water which passes through the earth.

Fig. 42, then, also shows why a tube tunnel, under these conditions, fails to rise, but settles instead; for, to rise, it would not only have to raise all the material included between *D E F G*, but also shear all the material along *D-E* and *F-G;* and this factor, even in material which has been deposited at the bottom of water, is of no small amount. In short, it makes no difference whether the water is filled with earth, or the earth is filled with water; the action of each is independent, and, while they may act in unison, they do not, and cannot, act as a unit.

The writer has not had any experience beyond the air locks; but,

Mr. Haines. for several years, he has been closely associated with those who have; and he understands that all the foregoing conditions hold good. He has seen, back of the locks, rings so badly cracked and distorted that it seemed doubtful if a line of compression could be drawn which would remain within the section of the ring. It appeared evident that, while the ring had carried a great load, it was then of most service in preventing erosion of the earth and maintaining the form of its arch.

The design of retaining walls will be considered next, since it is in that respect that a pressure formula is probably most used. The results at first glance are startling. It is known, however, that many walls have failed when apparently they should not; and the writer believes that his theory shows why. It has probably been noticed that, while the theory has been developed from a revolution of the mass of earth, the pressures have been computed as arch thrust. This is not inconsistent, as at first appears, for, to exert pressure against a vertical plane, the mass of earth must develop weight enough to cause fracture and revolution of the mass, since, if it does not, the bank will stand vertically of itself and exert no pressure. In all the cases thus far taken up this revolution can easily take place; because it is impossible to get a perfect contact between the earth and the supporting medium; the slightest movement of the mass, however, causes crushing of the material at the bottom, and the portion above its frictional angle of repose at once becomes an arch.

In the case of a retaining wall, the filling is placed from the bottom up, and, if well compacted, and if the wall is of sufficient section, there will be no chance for the mass to revolve. The shearing strength of the material will take up most of the weight, and there will be no arch thrust against the wall. This is shown clearly in many cases by the shrinking of the material away from the wall. If, however, the wall moves, by sliding on its base, even a little; or, if water, passing through the weep-holes, carries away a small amount of the material; or, if tide water be working behind the wall and settling the mass, the arching effect of the material comes into full play, and the wall is overturned from the top.

Two conditions, then, have to be provided for: the first tending to slide the wall on its base, caused by the revolution of the mass of earth. This force, at the instant the mass moves, is equal to the entire weight of the mass revolving around a point at one-half the height, as at *A*, Fig. 43 (*a*). The center of gravity of a hemisphere is at *B*, five-eighths of the radius from *C*. The path of its revolution is from *B* to *D*. There is, then, in the pressure diagram, Fig. 43 (*b*), a force varying from zero at *A* to a maximum at *E*, as shown by the triangle, *A E F*, with its resultant applied at *D*, in Fig. 43 (*a*), normal to the wall, and tending to slide it on its base. The height, *D-E*, is five-

sixteenths of the total height, and shows why, in experiments, the resultant is found at about one-third of the height. Mr. Haines.

There may also be, as has been seen, an arch pressure tending to overturn the wall, as shown in Fig. 43 (*b*) by the triangle, *E G H*. There are, then, two conditions, both of which cannot occur at once, but either is likely to occur; and the wall should be able to withstand either one; but, as at present designed, it seldom is. The modern, light, reinforced wall, as usually designed, and shown by Fig. 44, does, however, fulfill these conditions to a remarkable degree, for, to slide on its base, it would not only have to overcome the friction along its base, but, also, crush the material at the toe, *A B*, or shear the earth along a horizontal plane from *B*, and this toe is well removed from the fracture line, *C D*. To overturn the wall, also, from the arch action, it would not only have to lift the volume of earth, *C E F G*, but also shear the full height, *C E*, unless the soil fails by compression at *B*, which is possible, and should be considered in the design.

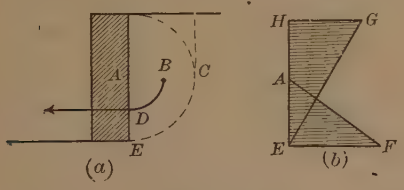


FIG. 43.

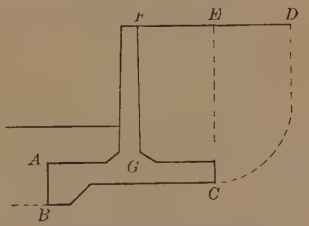


FIG. 44.

The writer believes there is but one case where it is impossible to compute the maximum pressure against a retaining wall, and that is where the material is stratified and sloping toward the wall. If the material be water-bearing, and overlying rock, or clay, it breaks away and moves toward the wall; and if the volume which would thus move, and the slope of the strata, could be determined in advance, then the maximum pressure could be determined. But these breaks often occur several hundred feet back, and the writer has known the break to occur even 1 000 ft. up the slope.

In practice, the conditions for maximum pressure do not often exist, but they should be considered. They exist sometimes, and the wall fails, this being usually attributed to faulty foundations, which is a rather poor excuse, and cannot often be proved.

There is no little literature on the subject of earth pressures, but the writer has found most of it of little value, being in most cases flatly contradicted by experience. Almost without exception, it is based either on Coulomb's theory of a wedge of greatest pressure—which the writer ventures to assert does not apply to material of granular structure—or, on a hydrostatic fluid with a weight equal to earth. This, the writer contends, cannot apply, because, the material

Mr. Haines. being insoluble, in order that it may act as a fluid, it becomes necessary that each particle be carried in suspension by the water; but, in order to do this, its weight must reduce to that of the water, and the hydrostatic pressure becomes that of water only.

A paper* presented before the Institution of Civil Engineers by the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., contains the results of a number of experiments, and, what is of more value, descriptions of walls which stood, and walls which failed, together with the conditions existing. The writer has found in this paper nothing directly contradictory to his theory, but much tending to confirm it.

In reference to the cracks on some 34 miles of deep-timbered trenches and tunnels, Sir Benjamin Baker stated that, on each side:

"The slope of these fissures was so uniformly at the angle of ½ to 1, measuring from the bottom of the excavation, that the resident engineer professed to be able to foretell with certainty where a building or fence wall, standing over the tunnel, would crack most."

He also stated:

"Assuming this ½ to 1 to represent Coulomb's line of least resistance, then the natural slope of repose of the material would appear to be 1⅓ to 1, which is considerably steeper than what it was in fact."

Trautwine, also, has called attention to the fact that some walls show bulging at about one-third the height, and that all walls tend to become vertical in spite of the batter. Neither of these facts can be reasonably explained by Coulomb's wedge of greatest pressure, or by a hydrostatic theory.

A few years ago, the writer built a retaining wall which failed; but, by all the existing theories, it should have stood. This wall was a bridge abutment at a river crossing. It was about 25 ft. high, and was founded on solid rock. The base was 42% of the height, and the face had a batter of 1 in. per foot. The wall had a section in the form of a triangle, with the apex 1 ft. above the back-fill. The parapet behind the bridge seat was 20 in. thick, and the back line was brought vertically down to an intersection with the triangle. The masonry was of cut stone, in courses from 30 to 16 in. thick, laid in cement mortar, and most of it in Flemish bond. The back-fill was placed in horizontal layers, by wheel-scrapers. This abutment stood perfectly for six months, until subjected to a 10-ft. rise of the stream, when a crack appeared in the bank, at a distance back somewhat less than half the height. The wall was checked up, and found to have moved outward 4 in., but it did not then incline outward. One week later the base was in the same position, but the top inclined 8 in. outward from its original plane, and some of the courses were shoved outward.

It seemed evident to the writer that the softening of the bank by the water had caused shear and movement of the wall, followed by

**Minutes of Proceedings*, Inst. C. E., Vol. LXV, p. 140.

the arching action of the dry material above and the overturning of the wall. This abutment was torn down, and rebuilt with a base of 50% of the height. In tearing down the wall, and removing the back-fill, the crack in the bank, with the characteristic curved fracture, could be traced nearly to the bottom, and no support was given the bank either during the removal or rebuilding of the abutment. As soon as completed and the back-fill replaced, the bridge was erected, and the writer has always been puzzled to know whether that abutment was actually holding up the bridge, or the bridge holding up the abutment, and also, by what line of reasoning the responsible parties arrived at the conclusion that a wall, which had failed utterly with a base of 42% of the height, would be safe (with a reasonable factor of safety) if the base were increased to 50% of the height. Mr. Haines.

In presenting this theory, which is believed to be new, the writer would state that, while the matter has long been in his mind, its actual preparation has been made rather hurriedly.

He would also call attention to the fact that the theory makes use of but two factors, namely, the weight of the earth, which can always be obtained, and the curved line of fracture, which he has observed so many times as to lead him to believe that it follows a natural law, as he has attempted to show.

Since writing the foregoing discussion, the writer's attention has been called to an article* by Mr. A. A. Steel, describing the apparatus used, and the results of some experiments on earth pressures.

The apparatus was of a size larger than usual, and was designed to test the pressures at different points in the height of a bank of earth. The results are interesting, and the following is abstracted from Mr. Steel's description:

"The diagram * * * gives the results of the experiments with damp earth. It will be seen that the pressure against the upper measuring board increases much more rapidly than that against the lower one. For this reason the series was discontinued shortly after the upper balances indicated a greater pressure than the lower ones. It was supposed that this was due to the earth clinging to the sides of the pit, like the sand in a molder's flask, and not settling freely. To avoid this the cohesion was destroyed by spreading the earth upon an asphalt pavement. Here, by the aid of convenient draft it was completely dried in about two weeks.

"The experiments were repeated with this dry earth, but again the pressure on the lower boards was less. This series of experiments required two days' time, and during the night the tangential component of the pressure on the upper board fell off about 60 lbs. This might have been due to a readjustment in the mass of the earth, but it seemed probable that it might have been caused by some meddlesome boy. To get a check on this and the difference in the pressure of the two boards, the series was repeated exactly, and a laborer engaged so that it could be finished in one day. The drop did not again appear * * *."

*_Engineering News_, Vol. XLII, p. 261, October 19th, 1899.

Mr. Haines. The point to which the writer would call attention is the fact that Mr. Steel had here a perfect example of the arching of loose material, for, the material being thrown in loose, the compression of the bottom material by the weight above it, and the friction of the mass settling against the boards, threw the upper portion into action as an arch, and, had the experiment been continued, some very valuable results might have been obtained.

As it is, his paper is a striking example of the extent to which one is governed by precedent, for, his apparatus being designed to accord with Coulomb's theory of a plane of rupture, his mind was unable to accept any results inconsistent with that theory, and the experiments were stopped.

Until investigators can bring themselves to make their deductions from observed facts, and not try and make the facts fit some predetermined conclusion, no really valuable results in any line can be obtained.

In conclusion, the writer thinks that the thanks of this Society are due Mr. Meem, for his paper; and, while he does not agree with the author's theory for pressure, he believes him to be perfectly right, in the case of sheeting, in placing the base of his pressure triangle at the top, instead of at the bottom. The writer also has the greatest respect for the author's system of timbering, for, from personal observation, he can state that it performs perfectly the functions for which it is designed, while some, at least, designed in accordance with the common theories, fails utterly, and more of it shows stresses far in excess of those for which it was designed.

Mr. Llewellyn. F. T. Llewellyn, M. Am. Soc. C. E.—This paper seems to be a very practical and valuable contribution to the scanty literature on this subject.

It is a subject with which the speaker is not very familiar, but, during the last twelve months, having had occasion to investigate the strength of mine timbers, he has found voluminous literature telling how to frame the ends of timbers, but hardly anything suggestive of a method of calculating their stresses.

The speaker believes that in the anthracite coal regions there are some practical "rules-of-thumb" (they can hardly be called formulas) in use by the mine foremen for computing the sizes of timbering at a depth of 500 or 1 000 ft., although these rules are largely modified by personal judgment. It is interesting to note that the one most commonly used in Eastern Pennsylvania is in the line with Mr. Meem's theory, although applied to depths very much greater than those in the trenches he mentions. This rule is as follows: In order to determine the safe distance between the rooms in an anthracite coal mine, it is calculated that the distance from center to center of rooms should be 1% of the distance below the surface of the ground plus five times the thickness of the overlying seams, the whole to be divided by two.

Mr. Llewellyn.

The main point of interest is that this formula contains two elements, the first being a function of the total depth, and comparatively insignificant, while the second, being a function of the thickness of the overlying seam, is the main factor, which would seem to support Mr. Meem's theory that the pressure does not increase at any rate in direct proportion to the total depth.

It has been stated that it was unfortunate that there had been practically no large tests of lateral pressures on pilings in trenches and elsewhere, and the speaker desires to state that the company with which he is connected is now arranging to conduct a somewhat extensive series of experiments in driving steel sheet-piling, and excavating to a depth of 50 ft. At present he cannot say what arrangements will be made for the dissemination of this information, but is satisfied that the results will be presented to the engineering profession as fully as possible.

Mr. Thomson.

T. Kennard Thomson, M. Am. Soc. C. E.—The author gives a formula, for calculating the strains on the bracing for open cuts or trenches, based on the angle of repose of the material, but he does not state how to find this angle, and, at the best, it could only be guessed at, for, while it would be easy to ascertain the angle for any loose material after it had been removed from the trench, it would seem to be impossible to determine it in advance of the excavation, as every foot underground is likely to disclose new conditions. Again, this assumed angle only holds good for dry material, as the author states, but there are very few cases of excavation where underground streams are not encountered; and water mains are likely to burst, and even a thunderstorm is apt to cause disastrous results.

It is quite true that it is sometimes safe to remove the bottom braces; in fact, the speaker has dug pits for elevated foundations in Brooklyn from 12 to 15 ft. deep without any bracing at all, but they were immediately filled with concrete, and, although no difficulty was experienced, it is not an example to be followed, even in Brooklyn, and certainly not in New Jersey.

In good hardpan, under air pressure, the speaker has excavated a depth of from 27 to 30 ft. without any bracing, but in many cases it would be suicidal to do this. Most New Yorkers must have seen banks of quicksand from which the water has been drained standing with a perfectly vertical face, and, when in this condition, of course, there would be no strain whatever on the bracing, and yet a slight disturbance would cause, and has caused, such banks to collapse without warning, and the daily papers have reported many cases of this kind where laborers have lost their lives.

It is quite true, as the author states, that there are many cases where his formula would not apply, so the only excuse for this discussion is the fear that some young engineer may overlook the excep-

Mr. Thomson. tions, and design his bracing by the formula, guessing at the angle of repose.

The experienced man will probably prefer to rely on his own judgment, as every case has to be studied by itself, and the more one has to do with earth or water pressures the more respect he has for their power and the less liberty he takes with them. There is probably not an experienced foundation man in the country who has not seen coffer-dam bracing collapse, sometimes at the bottom, sometimes half way up, and elsewhere.

In discussing fluid pressure, the author doubts if the full hydrostatic pressure would be found at the bottom. This is a subject which has been before the speaker for many years, in connection with deep cellars. He has put in cellar floors from 16 to 32 ft. below water, and the conclusion is that in some cases the full pressure does not occur, in many it does, while in others the pressure is much greater than the hydrostatic head would call for.

In pneumatic caissons in quicksand in New York City, where the caissons are sunk with as little disturbance of the ground as possible, it is found necessary to keep the air pressure almost exactly at the theoretical water pressure. Many want to calculate the pressure at from 90 to 100 lb. per cu. ft., including the weight of water and sand.

The speaker has seen a coffer-dam of 8-in. tongued and grooved sheet-piling driven several feet below the excavation before the digging started, braced with 12 by 12-in. walings, 5 ft. apart vertically at the top and less than 3 ft. apart vertically at the bottom, with 12 by 12-in. struts 6 ft. apart horizontally, and 30 ft. deep, where the waling deflected as a beam, the struts crushed 2 in. into the posts, and the timbers were pretty nearly knocked into kindling wood, the bottom caving in 2 or 3 ft.

Again, on the Canadian Pacific Railway in the Rockies, in 1885, there was a "clay" tunnel, high and dry, which was apparently all right until it was completed with heavy timber lining, when it suddenly shut up. Another effort was made, and 12 by 12-in. bracing was put in continuously instead of at intervals, and, as this tunnel also collapsed after completion, the company abandoned it and did not try again for more than 20 years, when another location was selected.

In open coffer-dam work, it is often better to use larger timbers spaced further apart than smaller sizes spaced closer, as more working room is left.

What the author refers to as sheet-piling has, in a great many cases, superseded the old-fashioned sheeting, as it is much better where it can be used, especially if the ground is wet or treacherous.

Mr. Jonson. ERNST F. JONSON, ASSOC. M. AM. SOC. C. E. (by letter).—This paper introduces a new problem in the theory of earth pressure, by

calling attention to the fact that the accepted theory cannot be applied to all cases, especially to those in which cohesion becomes an element in the stability of a mass of earth. Mr. Jonson.

It would be well if the author's empirical formula of earth pressure on sheet-piling could be verified by some more definite data than general experience, such as measurements of the stresses in the braces. This might be done by inserting hydraulic jacks with attached gauges, at the lower ends of the braces.

The author's simile of a wedge action seems to be unfortunate. If the pressure of an embankment were of this nature, and the pressure on a unit length of sheet-piling were $P = \frac{w\ h^2\ \text{tan.}\ c}{2}$, then, according to the same reasoning, the pressure on the upper half of the sheet-piling would be $P = \frac{w\ h^2\ \text{tan.}\ c}{8}$. Hence, it is seen that the average pressure on the upper half is one-half of the average pressure on the entire height of the sheet-piling. This implies that the pressure at any one point in the height of the sheet-piling is proportional to its distance from the top of the embankment. This, again, implies that the resultant of the pressure is located at one-third of the height from the bottom, which is contrary to the author's experience.

FRANCIS L. PRUYN, ASSOC. M. AM. SOC. C. E. (by letter).—The writer is deeply interested in this valuable paper as he has had considerable experience with almost all the various types of subway bracing that have been used in New York City. Mr. Pruyn.

Mr. Meem brings a valuable thought to the consideration of the engineering profession when he emphasizes the fact that earth pressures at the bottom of a trench need not necessarily be as great as those at the top. At the commencement of his paper he states that such conditions exist in dry ground, but, in the writer's opinion, he does not bring out with sufficient clearness the fact that such pressures can obtain in dry compact material only. In this the danger lies, for the author's experience has not only been in dry ground, but, in most cases, in artificially dried ground, as is the case in any paved city. The roofs of houses and the pavements of streets, together with the sewerage systems necessary to carry off rainfall, are artificial arrangements which tend to reduce greatly the quantity of water in such excavations.

It is a well-known fact that natural springs and watercourses dry up after a city has been erected over their former sites, for the simple reason that ground-water can no longer flow toward them. This artificial condition should be carefully borne in mind when the question of using Mr. Meem's theory of earth pressure in dry ground is contemplated.

In a city every rainfall is taken care of by drainage systems, so

Mr. Pruyn. that, unless a water main or sewer breaks, the engineer has little to fear from the increased pressure on the sheeting caused by the saturation of the ground.

In open country, no matter how dry the material may be when first excavated, saturation may take place after every rainfall, and bring about the hydraulic condition in earth pressure which the author's theory ignores.

What the writer desires to bring out is that, within certain fixed limits, Mr. Meem's theory is perfectly justifiable, but that those limits must be carefully defined and understood before any engineer attempts to design sheeting on that basis. The writer would define these conditions as those existing within cities with paved streets, and in dry material, such as sand or loam, in which the possibility of saturation by water is so remote that the contractor or engineer can afford to take the chance of the lighter bracing at the bottom of the trench.

Mr. Meem's experience, to a large extent, is based on the local conditions in the Borough of Brooklyn, which, of course, closely coincides with the theoretical conditions just stated.

The writer has had occasion to familiarize himself with the sheeting used on many parts of the present completed subway in Manhattan, as well as the Sixth Avenue subway, the Cortlandt and Fulton Street subway, and the Centre and Canal Street subways, now under construction. To show how limited is the application of Mr. Meem's theory of bracing, it is a fact that, with the exception of certain portions of the Brooklyn Loop on Centre and Canal Streets, where the conditions of a good sand and dryness of material closely approximate those of Brooklyn, it could not be used in any subway now being built in Manhattan. In all other cases the material is saturated and the hydraulic formula has to be used.

The author has done a great service to engineers and contractors in showing them how to save money in bracing and sheeting under certain peculiar conditions. For this the profession owes Mr. Meem its thanks. The writer's only point of criticism is that the limits of the application of his formula should be defined more distinctly, and should always be kept clearly in mind by the engineer who attempts to use it.

Mr. Shailer. R. A. Shailer, M. Am. Soc. C. E.—The speaker is exceedingly interested in this paper, but, during all the latter part of his life, he has been so much on the practical side of engineering that he must confess he is inclined to be cautious in reference to formulas and theories regarding earth pressures. Mr. Meem's theory is based on perfectly tight sheathing, which practically eliminates the trouble all have in getting competent workmen. With the men available, the timbering, in all cases, must be strong enough to hold the ground in case the sheathing is not tight. The theoretical discussion of such matters is

interesting, and, undoubtedly, real good will come from it, but, to the younger engineers the speaker would say: Mr. Shailer.

"Do not take chances, but be safe. You must not put in a 6 by 6-in. stick because 'theory' says it will hold, but double the size, and perhaps double it again, unless you are dead sure of every condition surrounding the work."

There is one point about the movement of earth in excavation which has been brought home to the speaker by experience, that is the advisability of getting the masonry in as soon as possible wherever the excavated material has a tendency to move. Swelling clay will stand oftentimes from two to three days without any perceptible movement. If timbered and left for a week or two, quite probably the timbers will be found cracked and buckled, be they ever so heavy; therefore, in clay tunnels, constant and steady progress of completed work is almost a necessity.

The importance of driving the braces tight in an excavated trench to prevent as far as possible the movement of earth or sand back of the sheathing, has already been brought out. In the speaker's opinion, this is a matter that deserves the closest attention, for, from the moment the movement of the material commences, the trouble begins.

H. P. Moran, Esq. (by letter).—In the construction of the Brooklyn Subway, with which the writer is connected, it was necessary to excavate a trench averaging 30 ft. deep and 5 000 ft. long. The bank was composed of sand and gravel with some few boulders. The bracing, designed by Mr. Meem, had the larger braces at the top and smaller braces when approaching the bottom. In no single instance has any failure of the lower and lighter braces been observed. Many instances of bending of the upper rangers, however, have been noticed. Mr. Moran.

In reconstructing the Hanson Place sewer, the trench excavated was 45 ft. deep, and there were seven sets of rangers; the larger ones being at the top. It was noticed that the top rangers had bowed at various places, the lower four rangers remaining perfectly straight. Great care was always taken to prevent any movement of the bank during the process of bracing.

It seems obvious to the writer that, under similar conditions of fill, the greatest pressure will always come at the top.

A permanent structure, such as a retaining wall, should be designed to withstand the maximum thrust of the earth. One should determine: First, the total pressure; second, its line of action; and third, its point of application. Obtaining the total thrust, it is evident that as the lever arm is increased, the overturning moment becomes greater. If the point of application be taken as two-thirds of the distance from the bottom, the maximum overturning moment will be obtained.

Comparing the thickness of a rectangular wall, designed, first according to Mr. Meem's method, and second, according to current prac-

Mr. Moran. tice as represented by Trautwine, it is found that, for a masonry wall, the thickness in the first case will be 46% of the height, no allowance being made for wall friction; in the second case it will be 35% of the height.

Fig. 45 shows graphically a wall designed according to Mr. Meem's theory, with allowance made for wall friction. Fig. 46 shows the same wall with a batter of 1½ in. per ft. Fig. 47 shows a wall having a thickness of 35% of the height (current practice), checked graphically by the same method. In Fig. 47 the resultant passes outside of the toe, indicating that the stability of the wall is questionable, and that excessive pressure would come upon the toe of the wall.

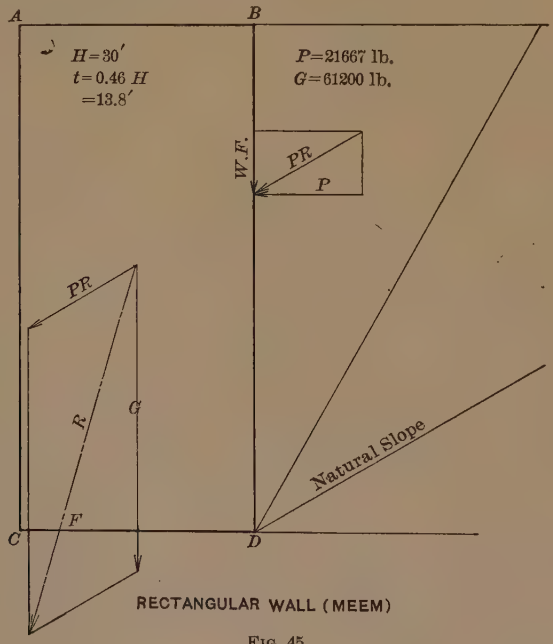


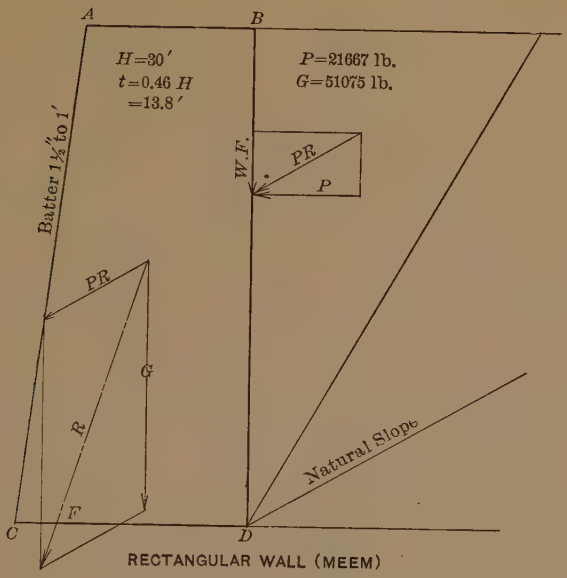
Fig. 45.

The numerous failures of retaining walls, 90% of which are supposedly due to improper foundations, can be readily explained when considered in the light of Mr. Meem's theories.

A retaining wall with buttresses presents a different problem—one analogous to that of a sheeted trench. Here the buttresses correspond to the braces, and the wall to the rangers. The wall, therefore, can be designed as a beam, in horizontal sections, to resist sliding and shearing. The overturning force is resisted by the buttresses.

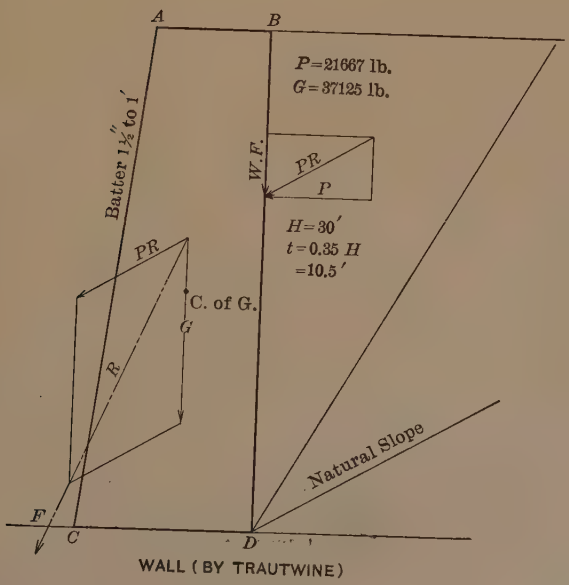
The pressure varying as the square of the height, and the point of application being two-thirds up from the bottom, gives a wall, triangu-

Mr. Moran.



RECTANGULAR WALL (MEEM)

FIG. 46.



WALL (BY TRAUTWINE)

FIG. 47.

Mr. Moran. lar in section, having the thickest part at the top and diminishing to zero at the bottom. Practically, it need not be so, and an average thickness could be taken, on the ground that the earth will not only arch itself vertically, but also horizontally, having its springing line at each buttress.

In the case of a reinforced concrete wall with buttresses, the increased strength required at the top could be easily obtained by spacing the horizontal reinforcements closer at the top, and increasing the spacing when approaching the bottom.

A reinforced wall with counterforts tied securely presents a similar problem, where the wall itself can be considered as a beam. As before, the spacing of the horizontal rods should increase as the bottom is approached.

From an analysis of the foregoing examples, the writer concludes that the total force acting cannot be determined by Coulomb's wedge of maximum thrust. This is based primarily upon the assumption that the surface of rupture is a plane. He considers that the surface of rupture is curved, probably parabolic in outline. Using this assumption as a working basis, the total pressure acting two-thirds up from the bottom will be less than the amount obtained by Mr. Meem's straight-line formulas.

Mr. Stern. EUGENE W. STERN, M. AM. SOC. C. E.—The practical experiences recounted by Mr. Meem are extremely interesting and valuable, but the formulas advanced by him do not appeal to the speaker, who still has faith in Coulomb and Weyrauch.

Mr. Meem's conclusions are based on experience in excavating and tunneling through a sandy material under almost ideal conditions. The street paving would prevent the earth below it from becoming saturated with rain water, and the small amount of water which might get through would be very quickly dispersed through the sandy soil, and would be just sufficient to "temper" it nicely.

Altogether different experiences have been obtained in going through a clay or loam soil, which would tend to hold and absorb water. A dry clay bank would stand almost vertically, and if excavations were made through it in perfectly dry weather, such as might obtain in the western arid portions of the United States, where practically no rain falls during the summer, it might be safe to carry out deep excavations with practically no sheeting or bracing whatever. But, as soon as the soil becomes saturated with water, the conditions are entirely different, and instead of its holding an almost vertical face in excavation, it will assume an angle of repose sometimes considerably less than 30 degrees. Now, the surface water cannot get into the soil all at once; it will have to work in gradually, the top layers becoming saturated first. The pressure on the bracing, therefore, would be greater at the top than at the bottom at first, but, this

seepage continuing, it would be found later that at the bottom the pressure was very much greater than at the top. Recently, the speaker had a case which illustrates this point admirably. An excavation for a trench was made in clay soil in dry weather, and the sides were almost vertical. As a precautionary measure, sheeting was put in, with cross-braces. Owing to the negligence of the contractor, only one line of these braces was put in, about half way up from the bottom. A heavy rain fell during the night, and the sheeting was pushed close together at the bottom and out at the top, showing very clearly that the pressure at the bottom was greater than at the top. Mr. Stern.

Another experience by the speaker was in connection with the abutment of a bridge which he was called upon to reconstruct. This abutment, supporting a fill of about 20 ft., was built in the summer, in dry weather, and the filling, consisting of clay loam, was completed also during dry weather. The abutment held up perfectly until the following spring, and then, the clay bank having been saturated with water, and no adequate provision having been made to withdraw this from behind the abutment, it bulged at about one-third of its height above the bottom, showing that the pressure at this point was greatest. This experience is not in accord with the author's formula.

The speaker, also, cannot agree with the theory advanced by Mr. Meem as to the pressure above the roofs of tunnels. He states that the less the angle of repose, the less the pressure will be upon the roof of the tunnel, so that a material like very fine sand, of which the angle of repose is, say, 20°, would cause small pressure, and a material in which the angle of repose is much greater, clay and loam soil, for instance, would cause a greater pressure. If, for the sake of argument, the material had no angle of repose whatever, or practically none—approximating then the condition of a fluid, the molecules of which are frictionless or nearly so—there would be still less pressure on the roof of a tunnel, according to Mr. Meem; with which contention the speaker cannot agree, as this is exactly the opposite of the facts. In the latter case, the pressure is the greatest that can possibly be obtained.

The late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., in a paper, read before the Institution of Civil Engineers many years ago, on the "Actual Lateral Pressure of Earthwork," mentions an interesting case in driving a heading for the Metropolitan Railway. He says:

"The ground consisted of sand and ballast, heavily charged with water, overlying the clay through which the heading was driven, at a depth of 44 feet from the surface. After the heading had been completed some months, the clay became softened to the consistency of putty by the water which filtered through the numerous fissures, and the full weight of the ground took effect upon the settings. Both caps and side trees showed signs of severe stress throughout the entire length of the heading."

Mr. Stern. This is an interesting example of a condition in which, at the start of the work, the angle of repose of the material being great, the pressure on the roof and sides of the tunnel was not large, but, as soon as the soil became saturated with water, and the angle of repose consequently became very much less than it was originally, the pressure increased enormously.

The speaker believes that the earth over the roof of a tunnel acts as an arch. The greater the angle of repose the material has, the more readily the earth arches, and the greater its strength. The less the angle of repose, the less readily it arches, and the less its strength.

In connection with earth pressures, it is rash to draw any broad conclusions from special experiences, for the reason that the conditions of the soil may vary, and the seepage of water has such an extremely important effect that great variations in pressures may result from these causes. A homogeneous clay in which there is no sand whatever will act differently from one in which there are pockets of sand, or which has gravel or sand overlying it, and entirely different from sand alone, or sand and gravel, and these materials when saturated with water will again act differently. Practical formulas to suit all these conditions would be an impossibility.

The observations of Mr. Meem as to pressures against braces, deduced from his own work, are no doubt correct, but these do not prove that the theories of Weyrauch and Coulomb are wrong. Theory, to be applied properly to a particular case in practice, must take into account all the conditions. Now then, what are the unknown elements in earth pressure?

Eliminating the question of water pressure in the soil, for the sake of argument, and having given the height of the bank, the only point that is uncertain is the angle of repose, and unless the angle of repose is known absolutely, of course, theory and practice cannot be made to agree.

The error most commonly made in applying theory to practice is the assumption that the angle of repose remains constant, no matter what the depth; this, of course, is not tenable. There is no reason why it should be constant, and every reason why it should not be.

In soil that has stood for ages in its original condition, the lower layers will be very much more compacted than the upper ones, by reason of the pressure of the earth above. If excavation is carried deep enough, it is found that the earth is almost in the condition of rock, it is so compactly consolidated. By applying sufficient pressure to a sample of earth, say in a hydrostatic press, it would be possible to make it almost as firm as soft rock.

The angle of repose, therefore, would increase with the depth below the surface in some proportion as the compactness of the soil

increased. This condition would apply, not only to clay and loam, but also to sand, when not under the water line. Mr. Stern.

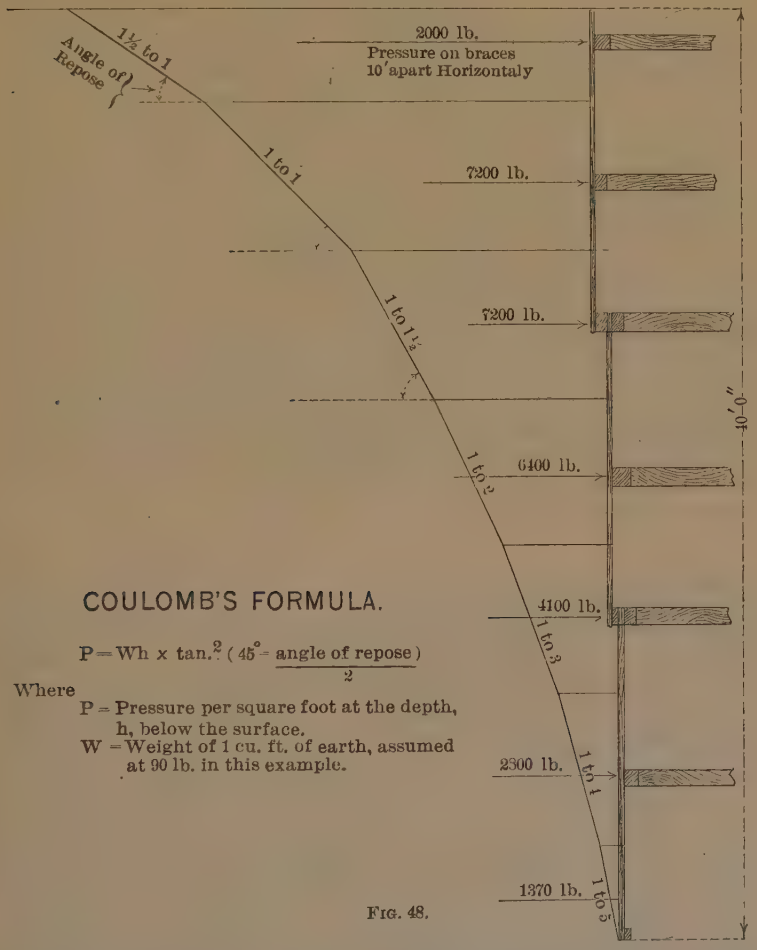

1½ to 1
Angle of Repose
2000 lb.
Pressure on braces
10' apart Horizontaly
1 to 1
7200 lb.
1 to 1½
7200 lb.
40'-0"
1 to 2
6400 lb.
COULOMB'S FORMULA.
4100 lb.
1 to 3
P = Wh x tan.² (45° - angle of repose / 2)
Where
P = Pressure per square foot at the depth, h, below the surface.
W = Weight of 1 cu. ft. of earth, assumed at 90 lb. in this example.
2300 lb.
1 to 4
1370 lb.
1 to 5

FIG. 48.

Take the case of an excavation through coarse sandy soil in which there is a very small quantity of loam (this is almost always the case). The soil below the surface is usually damp. At the surface, the angle of repose may be 1½ horizontal to 1 vertical. At a depth of 40 ft. below the surface, it may be 1 horizontal to 5 vertical. In clay soil it may be 1 horizontal to 3 vertical near the surface, if dry, and 3 horizontal to 1 vertical if wet, and 20 ft. below, 1 horizontal to 6 or 8 vertical.

Mr. Stern. Now, if the horizontal pressures are computed by Coulomb's theory, on this basis, for the same trench that Mr. Meem has given in his example, Fig. 7, the angle of repose varying gradually from 1½ horizontal to 1 vertical at the surface to 1 horizontal to 5 vertical at 40 ft. below, the pressures on the braces are found to be as shown in Fig. 48.

These pressures, with the exception of that in the top brace, which is less, agree so closely with those given by Mr. Meem that his practical observations do not prove that Coulomb's and Weyrauch's theories are wrong.

The question now arises: Of what practical use are the formulas advanced by the author? Are they sufficiently correct to be used generally? The speaker thinks not. In his opinion, they are of limited application, and—most important of all—the principles enunciated are radically wrong.

Mr. Christian. G. L. Christian, M. Am. Soc. C. E.—Mr. Meem is to be congratulated on his paper, relating to a subject upon which little is written. He has evidently given it much thought, guided by his valuable practical experience.

The speaker has in mind two retaining walls (neither of which was designed by engineers) varying in height from 5 to 15 ft. They were observed for more than twenty years, and, in that time, gradually failed in places, by bulging, usually at one-third the height, although occasionally the bulge appeared at one-half the height. The bulging in any particular place was always very slow, and was not sufficient to be noticeable unless several years had elapsed between observations. In the course of time, the parts of the wall where bulging had occurred collapsed.

The speaker once had occasion to place some extra braces across a trench, about 20 ft. wide and of the same depth, in a somewhat clayey soil, and instructed that they be placed pretty well toward the bottom. This did not agree with the ideas of the superintendent for the contractor, who claimed that the greatest pressure was near the top. This was a theory which the speaker had never heard advanced before, and did not believe in then, nor does he now; but he had a good deal of faith in the practical ability of the contractor's representative, and as it was very important that the banks be held rigidly in place, the safe course was adopted of putting in the additional timbers near the top as well as near the bottom.

The speaker does not agree with Mr. Meem's theory that the deeper the trench the lighter should be the lower bracing, although possibly that might be so, provided the trench is sheeted well and there are absolutely no voids behind it. Under those conditions, the author's argument is probably valid, but fails immediately on the appearance of voids, or very soon thereafter.

FIG. 1.—WEST HEADING, MORRIS AVENUE, WEBSTER AVENUE STORM RELIEF SEWER, BRONX BOROUGH, NEW YORK CITY

FIG. 2.—LOOKING EAST FROM WEST HEADING, MORRIS AVENUE, WEBSTER AVENUE STORM RELIEF SEWER.

The question resolves itself into whether or not it is practical to excavate in any soil and not leave voids, or opportunities for voids to occur behind the sheeting. The speaker thinks it is not. Mr. Christian.

It would seem that the sheeting on each side of a trench is subjected to the same thrusts as those on a retaining wall in the same position; therefore, as it is necessary to make the wall thicker at the bottom, it follows that the bracing should be designed to withstand the same pressures, and, therefore, must be heavier at the bottom than at the top.

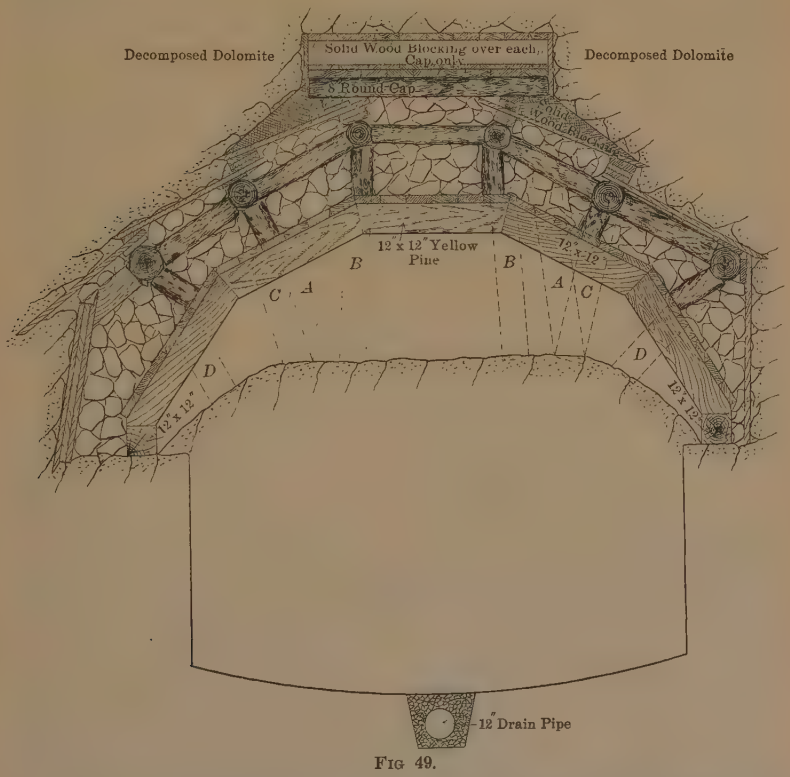


FIG. 49.

In the Borough of The Bronx, New York City, the Webster Avenue Storm Relief Tunnel Sewer is now being driven. Much of the tunnel work passes through a very much decomposed dolomite rock resembling clay. In such places the tunnel has been driven and timbered by the crown-bar method referred to by the author. In this work, a top heading was first driven, as shown by Fig. 1, Plate V. The cap and legs are shown in Fig. 49, the latter by the dotted lines, *A*.

Fig. 49 is an actual cross-section taken on this work. The two

Mr. Christian. center crown-bars are placed in position and held up by the posts, *B*, and then blocking and wedging are driven between the crown-bars and the caps, after which the legs, *A*, are removed. Transverse lagging is next driven above the crown-bars, as shown, after which the next two bars are put in position and held up by the posts, *C*. Transverse lagging is then driven above these bars in the same manner as before, the excavation underneath proceeding meanwhile, after which the two remaining bars are placed in position and held by the posts, *D*. The sides are then lagged; excavation is made for the wall-plates, which are then set to line and grade; and then the segmental arch is placed in position and blocked tightly against the crown-bars, the posts, *B*, *C*, and *D*, being taken out as the work advances.

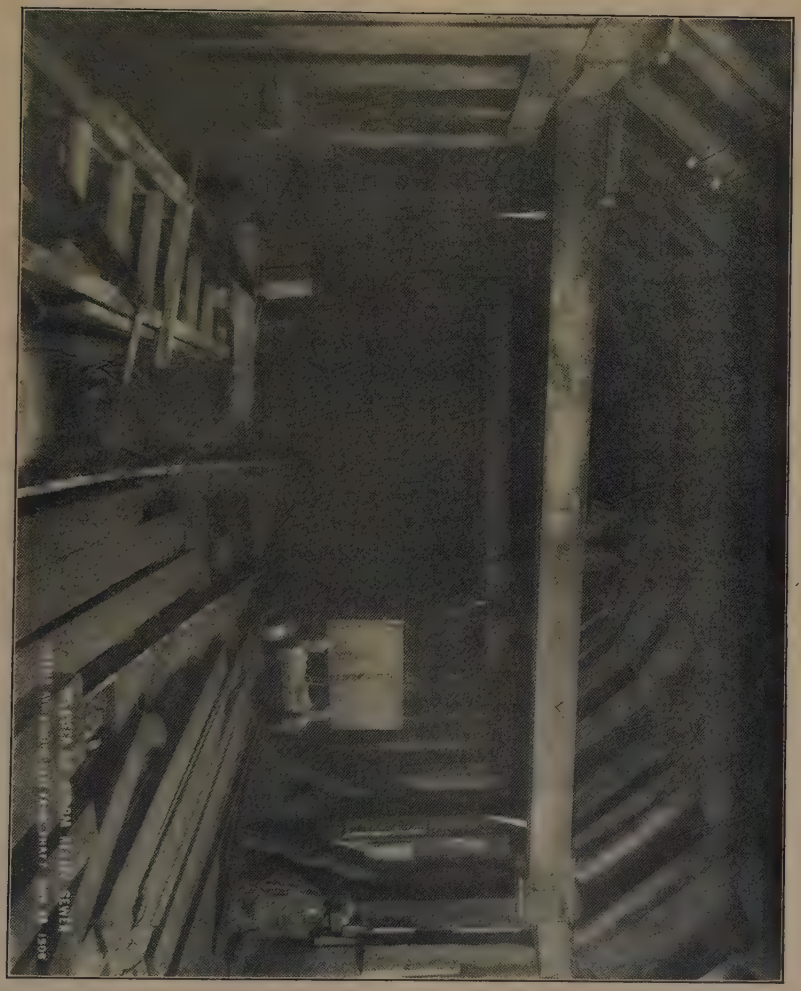
A good example of this work is shown by Fig. 2, Plate V. In the background may be seen the completed segmental arches, while in the foreground is shown much of the advance timbering just described, *i. e.*, the crown-bars above and their supports, the posts, *B*, showing on the left, and *B*, *C*, and the bottom of *D* on the right.

The bench is next excavated, and then the wall-plates are underpinned, where necessary, after which the work looks like that shown in Plate VI.

Mr. Hewes. V. H. Hewes, M. Am. Soc. C. E.—The speaker has become greatly interested in the several points brought out in the discussion, and would call attention to the general practice of miners in timbering. After having placed the timbers, the miners proceed to wedge them up so thoroughly that they fairly ring when struck with a hammer. By striking them in this way, they test them to see if the work has been properly done. Thoroughness in wedging is of the greatest importance, for, if there should be any slackness in the timbering, it would allow a movement to take place in the supported material. Should any movement occur, enormous pressure would be developed, often so great that, even if solid cribbing were placed in the workings, the timber would be crushed down.

Several years ago the speaker visited an old Mexican coal mine, then being worked by the fuel department of an American railroad. The timber in the rooms had been removed and the pillars robbed. This brought on what was termed a crush, or squeeze, and no amount of timbering could be placed in the workings to stop it. The only recourse was to run new entries, leaving a considerable body of coal between the workings and the new entry; even then, there was evidence of the movement by the cracking sound given out along the entry.

Evidence of the great pressure exerted by a mass of material which has been disturbed is often shown by the caving in of a mine. The speaker had occasion to go to the 200-ft. level of a mine, where a cave had taken place. The ore was soft hematite. In driving into the



WEBSTER AVENUE STORM RELIEF SEWER, BRONX BOROUGH, NEW YORK CITY.

broken ore at this level it had become so solid and compact that it was impossible to tell which was "unbroken ground" except by following the track rails, which were still in place. It was necessary for the miners to shoot it, in the same way as they would virgin ground. Mr. Hewes.

In an open trench or excavation, if not timbered, the ground in most cases starts to move at or near the surface, and fills up the trench, while that near the bottom may never move. The ground at the top has nothing to hold it in place and prevent its movement, while that at the greater depth is held in place by the material above it. To prevent any movement taking place, timbering is used; and Mr. Meem's experience, that the greatest pressure comes nearer the top of the trench, would not hold when any movement takes place at greater depth, for, in such cases, which often happen, much heavier timbering is required at the bottom than near the top.

E. P. Goodrich, M. Am. Soc. C. E.—This paper should be given much credit, but, at the same time, it is open to considerable adverse criticism. The author presents a number of exceedingly interesting and instructive examples illustrating actual practice in the bracing of trenches and tunnels, but his theories seem to be open to grave question both from theoretical and experimental standpoints. Mr. Goodrich.

It is a well-known fact that freshly cut masses of earth of certain kinds will stand for considerable periods with very steep slopes, but, in most cases, it may prove exceedingly dangerous to take advantage of this phenomenon, in an effort to reduce the size and quantity of timbering used to brace those banks. Where the earth is contained in a street, between cellar walls carried to fair depths, and is of a naturally porous nature, some allowance may be made without much risk; but, where work is carried along a side hill, for instance, in a clayey soil, a similar procedure might result in serious damage and loss. Consequently, it is the speaker's opinion that the author's several ideas and recommendations should be adopted only where conditions make them manifestly applicable, as approved by experienced engineers, and that they should not in any wise be considered as of general application.

In a few instances, in the speaker's experience, he has encountered earth pressures which were evidently greater near the ground surface than at lower levels. On the other hand, the opposite condition has been observed in a considerably greater number of cases, and numerous experiments of a very careful nature have shown the close agreement between fact and theory in this regard. These seemingly contradictory observations may be partly explained as follows:

When sheeting is driven along the side of a trench, the earth behind the sheeting is more or less loosened and otherwise disturbed; movements of greater or less magnitude may take place, in the form of slips, sprawling or crawling; cracks or voids may be formed, allow-

Mr. Goodrich. ing further movements to take place, etc. Such action may be very slow in culminating, but is almost sure to happen with lapse of time and variations in humidity. Furthermore, it is an observed fact that when large breaks occur, they are likely to take place along a curved surface, such as shown diagrammatically in Fig. 50. This fact is recognized by several authors of earth-pressure theories, and is the basis of the theory presented in the discussion of this paper by Mr. Haines.

Now, in dry soils, when the toe is prevented from sliding, a mass may crack away from the bank and tilt over at the top, turning about the toe, as shown in an exaggerated manner in Fig. 51. Such action would account for the cracks often found at various distances back from the edge of an excavation, though they may also be produced by local settlement, or by several other causes. It would also throw the heaviest stresses into the upper braces, and would tend to confirm the author's theories. On the other hand, if the toe is not so well fixed, and if the soil is more humid, a slip is likely to take place, as shown in an exaggerated manner in Fig. 52. This is the usual condition, and, obviously, would produce maximum stresses in the lower braces.

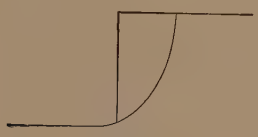


FIG. 50.

FIG. 51.

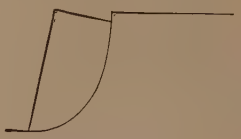


FIG. 52.

Again, many soils will "crawl" or appear to flow, acting like a viscous material. For such conditions, the theory advanced by the author would seem untenable, because both vertical and lateral pressures are then proportional to the depth. Damp clay is such a material, while dry clay may possess considerable cohesive strength. The speaker has seen tunnels driven through clay, and these have stood, for days at a time, for considerable distances and heights, without support and without change. In other cases, he has known the sides of similar tunnels to stand without material change of form, but to move bodily inward with such force as to crush to pulp the ends of heavy timbers. In another case, he was called to examine a tunnel and the nearby buildings, where the sides of the brick tunnel lining had been crushed inward, allowing a movement of the soil which endangered the foundations of even relatively distant buildings. The humidity of the soil had been greatly increased by the bursting of a sewer, thus producing unexpected conditions of stress which were quite extraordinary, but evidently also really possible. The two last

mentioned cases are explainable only by such theories as developed by Rankine; and those of the author would prove entirely inadequate. The latter must evidently be qualified most carefully as to the period of time which can elapse, the conditions of humidity, the kind of soil, etc. Mr. Goodrich.

This qualification is actually made by the author in several instances, thus showing the pertinence of the speaker's criticism.

On page 9, the author says, "where the trench does not have to stand too long." On page 19, he advocates maintaining "some sort of a bench at or near" the toe of the bank inside the sheeting. This seems to be inconsistent, when viewed in the light of his statement on page 6, that "the lower part of a trench may be left unsheeted." On page 13, he qualifies the application of his theories to clays, by saying that they "frequently develop pressures by squeezing or sliding horizontally, for which it is difficult to provide." On page 2, however, he has endeavored:

"to develop a practical basis in connection with which it will be possible at all times to effect an approximate reconciliation between the actual conditions of stability of earth and the theoretical formulas or resultants arising therefrom."

The qualifying statements introduced by the author seem to place his theory, also, among those which he condemns as not being able "to reconcile the theoretical with the practical conditions."

Again, on page 14, Mr. Meem speaks of a "frictionless material" having "an angle of repose" greater than zero. This is an obvious inconsistency, and when a zero angle is properly applied to the condition being discussed by the author, his seemingly logical deduction as to the weight coming upon the top of a tunnel would lead to an infinite value, which could not possibly obtain.

The author's deduction, on page 16, that earth will arch around a circular manhole with the same perfection at "indefinite depths," as close to the surface, does not seem compatible with the physical conditions which must exist in an arch formed in a granular mass. It is true that such materials will form effective arches over considerable spans, with certain sizes of grains and conditions of humidity, friction, or cohesion. For example, wheat will often arch completely across the ordinary elevator bins, and the effects on each other of loaded piles driven in soft ground show analogous action over distances of 20 ft. or more. Most earth-pressure experiments also confirm these observations, but to push, as far as the author has done, any deductions as to the effects of arch action, seems like exceeding the limits set by facts and sound theories. Because various large voids may have existed for long periods is not deemed a sufficient premise for all the conclusions reached. On page 15, the author does not give any definite

Mr. Goodrich. limit beyond which "the * * * arching effect is destroyed," and the arch action considered on page 12 can take place only over certain spans, the limiting value of which is not stated. This seems to be a most important omission, except that it is believed by the speaker that conditions are so variable that no precise limit can possibly be set in either instance. It would thus appear necessary in the first instance almost invariably to use for shafts such "bracing" as would be used "in an open trench," as suggested by the author for large shafts.

The author's reasoning on pages 13 and 14, as to the effects of the angle of repose on arch action, is not at all clear. Furthermore, it seems to be based on entirely wrong assumptions, when the author states that there can exist a "tendency to slide along the angle of repose," using this phrase as he does synonymously with that of the "natural slope of the earth," as on page 2. The experiments made by the speaker, and reported to the Society in his paper on "Lateral Earth Pressures and Related Phenomena,"* are believed to have demonstrated the existence of an angle of internal friction along which sliding will take place, which angle is usually very different from the angle of natural surface slope. Consequently, the author's work must be at least so modified as to include this angle of internal friction.

The ideas set forth by Mr. Haines approach nearer to those of the speaker than those of any investigator on this interesting subject. The speaker, also, has measured numerous slips—natural, and made by experiment—but has not found the hemispherical surface as accurately followed as one might infer was always the case, from Mr. Haines' descriptions. Arch action along lines which closely approximate circles is usually present in most soils, as soon as any movement takes place, and in this special point, such earth masses act like solids. But vibration, changing humidity, natural settlement, and lateral pressures bring about further movements among the particles composing the mass, so that an action closely analogous to the slow flow of a viscous mass also often takes place. Mr. Haines, however, takes no account of these possible changes of condition with lapse of time, although this is absolutely necessary in a complete theory. To be sure, a retaining wall which is stable under the author's or Mr. Haines' assumptions will be apt to stand under conditions like those usually assumed, but it does not seem to be good engineering to waste material in securing a needless excess of stability, as would seem to be the case in designing by either of the two theories mentioned.

As opportunity offers, the speaker is working out the combined theories of the several actions which occur in masses of earth, and hopes some time to present them for discussion. He doubts Mr. Haines' analysis on page 30, which includes simply the volume and

* *Transactions*, Am. Soc. C. E., Vol. LIII, p. 272.

surface of a sphere. If the earth mass were supported on a shaft through the point, *B*, of Fig. 28, the reasoning might apply, but the mass which eventually moves actually rests with considerable weight on the earth beneath, which remains in position. Consequently, other forces besides pure shear enter the problem, and the speaker believes that Mr. Haines' explanation of the spherical surface of slip is not sound. Mr. Goodrich.

As to his analysis of the stresses due to the arch action, as he supposes it, attention should be called to the fact that the "center of gravity," as he expresses it, does not coincide with the center of length, as shown in Fig. 32, so that the stress, *T*, as found by him, is smaller than it should be according to the method he has adopted. There is also a grave doubt in the speaker's mind whether it is even approximately correct to assume that the lateral stresses against trench sheeting are such as would be produced by a series of arch rings between which no friction exists. It would seem rather more nearly according to reason and fact to assume a single ring of some thickness, just above the surface of fall, which ring is assumed to carry the weight of all the earth above it. This supposition would explain the shrinkage away from the sheeting near the surface, which the speaker saw on one occasion, coexistent with evidences of considerable arch action near the bottom. It also explains the concentrated pressures observed by the speaker on several occasions, and gives a reason for the fact that practically no pressure is found at the top of the sheeting, where it should be a maximum, according to the theories of the author and Mr. Haines.

Time, however, modifies most things, and it would not seem improbable that such concentrations would eventually distribute themselves, more or less, according to some law of variation of pressure. Such a readjustment of stress (described later) has actually been observed by the speaker.

An entirely satisfactory earth pressure theory has still to be evolved, and more phenomena can be explained on the assumption that earth acts like a viscous, partially elastic material, than on any other supposition. This hypothesis explains all the author's observations; explains the phenomena described above; and also throws light on the change in the position of the point of maximum thrust and the distribution of pressures observed by the speaker during one of the many earth pressure experiments he has made. The conditions and methods of test and analysis are given in the speaker's paper on Earth Pressures, previously mentioned. The initial observations showed a roughly triangular distribution of stress. A special condition was soon brought into play by a sharp thaw in the material behind the wall, and a considerably increased and practically concentrated load was observed near the top of the embankment. The point

Mr. Goodrich. of application of this concentrated load then appeared to move slowly downward and was finally lost in the slightly parabolically distributed continuous load.*

While Mr. Meem is a most successful practitioner, and has done some very exceptional work, his theories with regard to earth pressures are not considered as well worthy of praise. It seems to the speaker, rather, that much more work must be spent on the problem before it is entirely solved, and that other lines than those followed by the author will lead to more consistent results. What might be called "Rankine's theory applied to a viscous granular material, wherein angles of internal friction are used," appears to be the most profitable course along which to investigate earth pressures and related phenomena.

Mr. O'Rourke. J. F. O'ROURKE, M. AM. SOC. C. E.—The speaker has had a great deal of trouble in trying to do timbering work according to theory. Mr. Meem's conclusions are largely according to his experience in dry, sandy and gravelly material. It is only necessary to examine his diagram, Fig. 7, in which the pressures are indicated by parallelograms, with the largest parallelogram on top, to commence at once to find fault with his theory.

It is only necessary to consider that these pressures are transmitted through the material to show that it is impossible, at the top of an earth strut, so to speak, to get the greatest pressure, or a sufficiently great pressure, to require 12 by 12-in. timbers. If there were such a pressure as that, the material itself would rise, and the pressure would disappear. The fact is that in most cases this top timber may be taken out (barring, of course, strains which may be caused by other things than the pressure of the earth).

The real trouble with timbering, and all this protection of excavation, is that where there is something besides dry sand or gravel, there is water to be dealt with, and then the material has varying friction, and, of course, varying pressures.

The speaker had cases on Park Avenue, New York City, where 12-in. I-beams were bent. These beams were set vertically, and were perfectly straight until the ground had become more or less wet, and commenced to take a totally different angle of friction, thus becoming what is called "heavy."

With the statement that the pressures are the reverse of those caused by water, the speaker cannot agree at all. His experience is that it is almost impossible to drive sheathing beyond a certain depth because of the great pressure that develops the farther it is driven, and the fact that the timbers incline inward at the bottom owing to the increased pressure. No one who has done much sheet-piling in excavations, particularly where there was water, will contradict that statement. It is a universal condition.

* *Transactions*, Am. Soc. C. E., Vol. LIII, Figs. 11 to 16, p. 278.

Mr. O'Rourke.

The general question of experiments in earth pressures and their conditions which will reduce the subject to a science, is an admirable one. The speaker's opinion, however, is that such experiments would have to be continued for a thousand years in order to obtain something in the nature of a formula, but that formula would have a great many constants. There might be a number of what might be described as arbitrary, unknown quantities, but having had a thousand years' experience, it is possible that one might learn to use them. However, notwithstanding the great success in the use of formulas, up to the present time, the better plan is to stick as closely as possible to big timber, no matter what the formula indicates.

Mr. Nichols.

O. F. Nichols, M. Am. Soc. C. E.—In dealing with what may be called non-fluid earth, a semi-dry or quite dry material, not like a fine sand which will run, but more like the sand and loam found in New York City and on Long Island, there is much of value in the suggestions made by Mr. Meem.

The angle of repose is determined by allowing material to run free in the open air; but, when it is flowing on itself, and has an angle of internal friction, so to speak, there is no doubt that the friction of the material tending to slide downward would set up some other angle of repose, or of partial stability, greater than the normal angle of repose and corresponding more nearly with the angle which Mr. Meem establishes as bisecting the complement of the angle of repose. Just what this angle is, is indeterminate, but, assuming that material does act in this way, the speaker has no doubt that Mr. Meem is right, that there is some arching effect with the skewback on one side which may be taken as the angle of repose, and on the other side by the sheeting of the trench.

This arching effect produces great pressure on the sheeting at or near the surface of the ground, and correspondingly relieves the pressure on the sheeting at points below, until, at the bottom of the sheeting, it is very often possible to undercut the sheeting, and to do this undercutting for long distances, and even to a great depth.

This arching effect, so far as it obtains, necessitates the use of heavier timbers at the top and lighter timbers at the bottom of the trench, and, where the material is not too wet and is not allowed to stand too long, so that water, the atmosphere, and other agencies can act on it, it is quite possible to leave the sheeting out for certain distances near the bottom of the trench. Constructors take advantage of this fact, for there is very seldom any shoring at the bottom of sheeting under the conditions assumed.

Engineers, however, must err if possible on the safe side, and it is probable that the old "angle of repose" conditions will still be used in earthwork calculations, mainly because it must be assumed that the worst conditions may prevail.

Mr. Nichols. A man—who, of course, was not an engineer or in any way connected with engineering—recently asked for a discrimination between men like the late Professor Bartlett, of West Point, the engineer of to-day, and the contractor. The answer should be that men like Professor Bartlett develop the laws by which the forces of Nature act on material, the engineer applies these laws, and the contractor adds to the knowledge of the engineer the saving grace of common sense, and, particularly, introduces the element of commercial values.

When, therefore, engineers become successful contractors, the combination should, at least theoretically, redound to the great good of the community. The questions and solutions which Mr. Meem offers in his interesting paper are especially those with which engineering contractors should be familiar, and it is to be hoped that such men will take part in this discussion, for they, like Mr. Meem, have fought out engineering battles in close contact with the materials themselves, with their ears close to the ground, and they are men who measure problems by the laws of profit and loss as well as by those of engineering skill and efficiency.

The work in this field which Mr. Meem has done in Brooklyn, in supporting nearly a mile of elevated railway in full operation, in addition to the surface of the street and one of the busiest sets of trolley tracks in the world, and the work he did in restoring the grade on the Brooklyn end of the Battery Tunnel, certainly mark him as an expert in engineering construction, and his paper should be of great value, especially in bringing out discussion.

Mr. Gahagan. Walter H. Gahagan, M. Am. Soc. C. E.—Those who have recently passed along Fulton Street and Flatbush Avenue, in Brooklyn, N. Y., cannot but admire the skill with which the subway work on these thoroughfares has been conducted. Credit for this work must be given to Mr. Meem, Chief Engineer for the contractors. The elevated railroad structures on these streets, from the City Hall to the Long Island Railroad Station, have been supported on temporary columns, for the full length of the excavation, and there have been no slips or settlements of the columns, and no interruption in the use of the tracks of the surface or elevated railroads.

Mr. Meem discusses pressures in dry, homogeneous earth, which conditions were found by him on this work, but, if hydraulic conditions are introduced, it becomes more difficult to determine the effect on the materials to be handled. The variety of materials is great, in all cases; they may be homogeneous, or may be mixed in any proportions, and may run through all gradations from dry to very wet.

The author does not intend that his formula for bracing, etc., shall be applied to all these various conditions, but only to cases where the material is reasonably dry and homogeneous. Under other conditions, the speaker would be inclined to follow Mr. O'Rourke's ad-

vice and "stick close to big timber," and, further, in cases where there is much water in the soil, such timber should be used from close to the top to very near the bottom of the trench. A special study of the conditions is necessary in each case, and the speaker does not believe that any general formula for earth pressures is practicable. Mr. Gahagan.

MILO S. KETCHUM, ASSOC. M. AM. SOC. C. E. (by letter).—While this paper is a valuable contribution on the practical side of the subject, the theoretical investigation is open to criticism. In the discussion of the subject of pressures in masses of earth, engineers often confuse actual conditions and freely condemn theoretical investigations because the formulas deduced for special cases will not give consistent results for all conditions. All theoretical investigations of the pressures of earth are based on the assumption of a granular mass without cohesion, the particles being held together by the friction of the grains on each other, the mass being of indefinite extent. Where the cohesion is sufficient to hold the mass in position, the theoretical formulas do not apply, and where the mass does not extend indefinitely back from the surface, as the grain in a grain bin, the Mr. Ketchum.

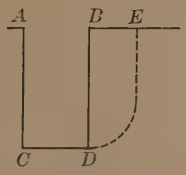


FIG. 53.

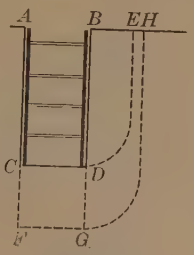


FIG. 54.

general formulas must be modified. The following solution of the problem of earth pressures on the sheeting of deep trenches is offered:

If a trench be excavated in earth in which there is cohesion, the sides will stand vertically until the soil loses its cohesive power, or, if the trench is excavated rapidly, a depth, *B D*, Fig. 53, is reached where the weight of the earth will overcome the cohesion and the friction, and the trench will cave in. When the slip occurs, the cohesion is practically destroyed in the mass, *B D E*, and if the factor of cohesion is known, the curve, *D E*, can be calculated.*

Now, if sheeting be placed in the trench and properly braced, the mass, *B D E*, will be held in place, and there will be no active pressure on the sheeting until the depth, *B G*, Fig. 54, is reached, where the weight of the mass, *B G H*, is sufficient to overcome the cohesion and the initial stresses in the bracing. It will be found that *B H* is not much larger than *B E*, which shows that the mass, *B E D*, is held

* Merriman's Retaining Walls, page 16.

Mr. Ketchum. up mainly by the friction of the earth on the sheeting, *B D*, and the surface, *H G*.

The pressure on the sheeting in a trench in a cohesive soil, then, is seen to be due to the material between the sheeting and the surface, *H G*. The problem of the calculation of the stresses is then essentially the same as the calculation of the stresses of a granular material in a deep bin.*

In Figs. 55 and 56, *C A* is the sheeting and *D B* is the surface of cohesive rupture, assumed to be vertical.

Let *W* be the weight of the wedge, *C A E*, 1 ft. thick; *P* be the total pressure on the wall; *R* be the reaction of the material below the plane of rupture, *A E*; *x* be the angle of rupture; μ be the coefficient of internal friction of earth; μ' be the coefficient of friction of earth on the sheeting, *C A*; and *O* be the center of gravity of the wedge, *C A E*.

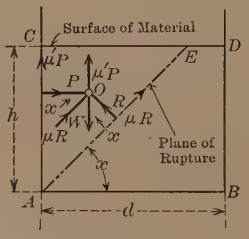


FIG. 55.

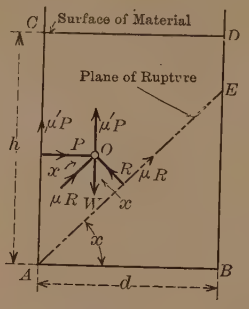


FIG. 56.

In Fig. 55 the material in the wedge, *C A E*, slides on the plane of rupture, *A E*; the weight of the wedge, *W*, is held in equilibrium by the pressure, *P*, the reaction, *R*, on the plane, *A E*, the friction, $\mu' P$, on the wall, and μR on the plane of rupture.

Now, resolving the forces parallel and perpendicular to the plane of rupture, *A E*, and solving for a maximum value of the angle of rupture, *x*, we have

$$\tan. x = \mu + \sqrt{\mu \times \frac{1 + \mu^2}{\mu + \mu'}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

and

$$P = w \times \frac{h^2}{2 \tan. x} \times \frac{\tan. x - \mu}{1 - \mu\mu' + (\mu + \mu') \tan. x} \quad \ldots\ldots\ldots\ldots \quad (2)$$

Equations 1 and 2 give the pressure on the side, *C A*, as long as the plane of rupture, *A E*, cuts the surface of the material.

In Fig. 56 the plane of rupture, *A E*, cuts the side, *D B*, and Equations 1 and 2 will not apply.

* "The Design of Walls, Bins and Grain Elevators," by Milo S. Ketchum, The Engineering News Publishing Company, New York.

Mr. Ketchum.

Equating the forces parallel and perpendicular to the plane of rupture, $A\ E$, in Fig. 56, and solving for a maximum value of the angle of rupture, x, we have*

$$\tan. x = \sqrt{\frac{2\,h}{d} \times \frac{1+\mu^2}{\mu+\mu'} + \frac{1+\mu^2}{\mu+\mu'} \times \frac{1-\mu\,\mu'}{\mu+\mu'}} - \frac{1-\mu\,\mu'}{\mu+\mu'} \quad \ldots\ldots\ldots \quad (3)$$

and

$$P = w \times \frac{d}{2} \times (2\,h - d\tan. x) \times \frac{\tan. x - \mu}{1-\mu\,\mu' + (\mu+\mu')\tan. x} \quad \ldots\ldots \quad (4)$$

It will be seen in Equations 1 and 2, 3 and 4 that the pressures vary as $\frac{h}{d}$. These equations are complicated, but special solutions may be made as follows:

In Fig. 57 the writer has plotted values of $\frac{P}{d^2}$ for different values of $\frac{h}{d}$ for "earth" weighing 100 lb. per cu. ft., angle of internal friction, $\phi = 31°$, $\mu = 0.6$, and angle of friction of earth on the sides $\phi' = 31°$, $\mu' = 0.6$, P being the total pressure on the wall 1 ft. long for the depth, h. To calculate the pressure on a square foot at a depth of 50 ft., calculate P for $h = 49\frac{1}{2}$ ft., and P for $h = 50\frac{1}{2}$ ft., and the difference between the two values of P will be the pressure per square foot at the required depth. In this manner the writer has calculated, in Fig. 58, the pressures per square inch for different depths for earth where the break, back of the wall, is 10, 20, and 30 ft. Pressures for other widths may be interpolated from the diagram. As an example, the pressure for a width of 5 ft. and a depth of 20 ft. is one-half the pressure for a width of 10 ft. and a depth of 40 ft.

For example, if the ground breaks 10 ft. back from the sheeting, the pressure at the depth of 50 ft. will be 4 lb. per sq. in., or 576 lb. per sq. ft. Of course, this pressure will not be developed until the excavation has extended far enough below this depth to destroy the cohesion. With a width of 5 ft. and a depth of 50 ft. the pressure will be one-half the pressure with a break of 10 ft. at a depth of 100 ft., or one-half of $4\frac{1}{2}$, or $2\frac{1}{4}$ lb. per sq. in., or 360 lb. per sq. ft.

The pressure on the mass below the given depth will be found by subtracting the weight supported by the walls from the total weight. For a break, back of the wall, of 5 ft. and a depth of 40 ft., the pressure on the side, from Fig. 57, will be $P = 375 \times 25 = 9\,875$ lb., or 19 750 lb. on both sides. The weight of a section of earth 1 ft. long will be $5 \times 40 \times 100 = 20\,000$ lb. The load supported by the sides will be $19\,750 \times 0.6 = 11\,850$ lb., making the pressure on the bottom $20\,000 - 11\,850 = 8\,150$ lb.

In the practical application of this solution, diagrams, such as Figs. 57 and 58, should be constructed for the weight, angle of repose,

*For the derivation of these formulas see the writer's book, "The Design of Walls, Bins and Grain Elevators."

Mr. Ketchum. and angle of friction of the material in which the excavation is made. The center of pressure will be at approximately one-half the depth for deep trenches. If quicksand is encountered, the pressure will be more nearly that given by the retaining-wall line. For a short trench, the horizontal projection of the break is a curve, but, for a long trench, the break will be approximately a straight line.

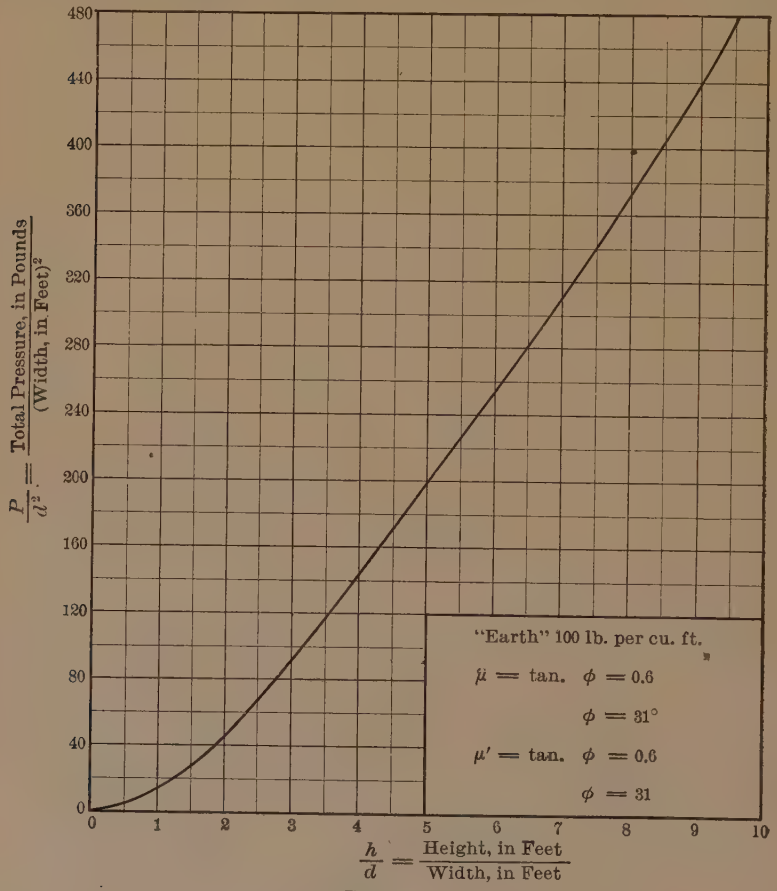


FIG. 57.

The foregoing solution gives results consistent with observations, as stated by the author, and is based on reasonable assumptions. Of course, it cannot be expected that any solution of the problem will give results which are more accurate than the data. It is the writer's opinion that, if engineers were more familiar with the theoretical investigations of retaining walls, there would be less objection to such investigations.

Mr. Ketchum.

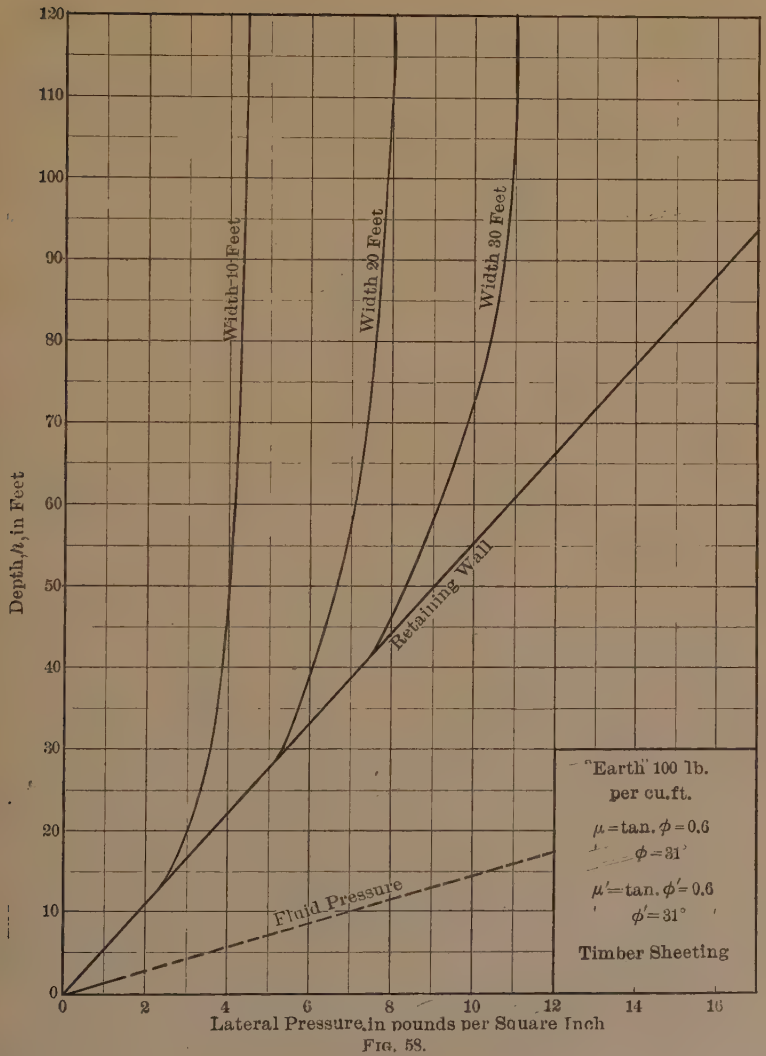


FIG. 58.

Mr. Marsh. CHARLES F. MARSH, M. AM. SOC. C. E. (by letter).—Mr. Haines states that the center of pressure in Fig. 43 (*a*) is at *D*, five-sixteenths of the height of the wall from *E*. He considers, also, that the pressure intensities will vary uniformly from a maximum at *E* to nil at *A*, as shown in Fig. 43 (*b*).

It seems very probable that the last assumption is correct, but, if this is the case, the center of pressure must be at one-third of the height, *A E*, from *E*, as the position of the center of gravity of the section of material, *A C E*, Fig. 43 (*a*), does not affect the position of the center of pressure, which is entirely controlled by the distribution of the pressure intensities along *A E*.

Mr. Haines is to be congratulated on his most excellent and convincing treatment of the subject of earth pressures, and the writer believes he has gone far toward giving a true solution of this most difficult problem.

Mr. Cranford. F. L. CRANFORD, ESQ. (by letter).—In the discussion of this paper, Mr. O'Rourke, among other criticisms, states that he thinks it wise to stick to big timber. It is possible that Mr. O'Rourke misunderstands Mr. Meem's intent. Mr. Meem's statement of the principles of timbering and excavation is the statement of an engineering principle. His theory was not advanced as an economy, and, while it sometimes provides an economical method of bracing an excavation, it is not discussed by the writer from an economical point of view. The accepted rules are old, and, judging by the writer's experience, they are wrong. A few practical illustrations, which have come within the writer's experience, will be given: Considering only the place, on the vertical sides of an excavation, where the greatest strength of timber work is necessary, this has always been found to be near the top of the excavation. It is very necessary, when building a large structure in a deep excavation, to know exactly where braces may be placed to sustain the load against them; for instance, in constructing large sewers, having an outside diameter of, say, 18 ft., in an excavation 30 ft. or more in depth, it is the general rule to build the invert of the sewer, up to the springing line of the arch, beneath the lowest brace, after which, that brace is removed, the arch centering is set, and the arch is turned. No special precautions are taken to back up the sheeting between the top of the invert and the sheeting of the trench, and, though it is the usual practice to back-fill this space before removing the lower braces, the writer has never known an instance where the removal of those braces was followed by any accident, while it would seem to be beyond controversy that, if those braces carried any considerable pressure, it would be necesary to fill the space between the invert and the sheeting with concrete or brickwork before removing them, it being apparent that, unless such a precaution were taken, the sheeting would spring inward to a greater or less degree,

depending on whether the back-filling had or had not been done. This initial movement of the sheeting would not stop under such a load, as would be the case if the accepted theories were correct. Mr. Cranford.

In laying a 48-in. cast-iron water main, some years ago, in a trench about 35 ft. deep, the material being sand with about 4 ft. of water in the bottom, it was found necessary to place the braces 6 ft. apart, from center to center longitudinally, the rangers being about 4 ft. apart vertically. The pipe was lowered to the bottom of the trench "end on," because at the bottom the lower set of rangers could be left with braces 3 ft. apart, from center to center, thus giving room to lower the pipe on its side in the bottom of the trench. On this pipe line (which, by the way, was 13 miles in length, and the trench from 10 to 15 ft. in depth), the scheme of bracing contemplated braces 3 ft. apart. This permitted the lowering of a 12-ft. pipe through the timbers. The material being sand, it was frequently found necessary to put in an intermediate brace. When lowering the pipe this intermediate brace of the top ranger was knocked out, the pipe was lowered below it, the brace was replaced, then the next intermediate brace was knocked out, and so on. After the pipe was laid, it was seen that the rangers were badly sprung, but the top ranger was always sprung twice as much as the second ranger, and the second ranger was sprung twice as much as the bottom ranger. The writer has seen such a trench in which the rangers were sprung in this way continuously for more than 2 miles.

In laying a pipe line around the Spring Creek Reservoir of the Albany Water-Works, some years ago, the trench was from 12 to 14 ft. deep, and passed through a stiff clay, the clay being in layers from several inches to 1 ft. in thickness, with very thin layers of quicksand between them. The layers of quicksand may have averaged $\frac{1}{8}$ in. in thickness, and usually carried a little moisture. The trench had been completely excavated, the excavated material being piled on one side, when it was noticed that the whole bank, on the side on which the excavated material had been piled, was sliding on one of these layers of quicksand about 3 ft. above the bottom, in fact, it had moved 3 in. when first noticed. An attempt to brace the trench was immediately made, and several hundred feet of it were saved, but before night a large amount of it, perhaps 400 or 500 ft., had moved so far into the trench that there was no hope of saving it, and next morning the excavation for this distance was entirely closed.

In excavating a high bank of earth, if the old accepted theories of earth pressures were correct, it would be reasonable to expect, when such a bank became so nearly vertical as to fall, that it would slump off, the weight crushing the lower layers of earth, and causing the top of the bank to sink. As a matter of fact, the writer has never seen a bank fail in this way (excepting sand), but hundreds of times he has

Mr. Cranford. seen such a vertical bank turn, as on a pivot at the bottom, and fall without breaking its form until it struck the ground at right angles to its previous position, showing that, had its top portion been prevented from falling out, the bank would have been supported.

Mr. Paige. JASON PAIGE, JUN. AM. SOC. C. E. (by letter).—The writer has read, with much interest, this paper and also the resulting discussion, especially that by Mr. Haines. Mr. Meem applies an angle of repose to the material, while Mr. Haines works on the assumption that a bank, as it exists in cases of open trench work, is not a granular mass, but one having a great many of, if not all, the characteristics of an elastic body. With the latter view of the case, the writer agrees, but he finds that at a certain point his investigations lead to a result, different from that reached by Mr. Haines, but which would still seem to agree with conditions found in practice, namely, that the greater pressure on bracing in deep trenches is found at a point about one-third of the depth of the trench from the top.

First, as to the condition of the material through which a trench is to be cut: A few simple examples will show that the condition of a material which will stand excavation to a certain depth without failure is far from granular.

Taking first the case of a load of clay loam which has been excavated and is undoubtedly in a granular state, for, in that condition, it does assume a certain angle of repose: Dump this load of clay loam on the ground, and it is of such consistency that a pick, due to its weight, will sink into it. Now allow this pile of clay loam to remain undisturbed for, say, four or five years, and then try the pick again. It will now be found that, not only does the material resist the dead weight of the pick, but it will also resist to a considerable extent a blow delivered with the pick, when both externally applied force and impact are present.

Now, to return to the freshly dumped pile of loam: If, from this pile, in some manner, one could take a mass 1 in. square and of some unit length, it would be found, if this mass were suspended by a clamp applied at the top, that the material possessed practically no qualities in tension, its ultimate strength not being high enough to sustain any large percentage, if any at all, of its own dead weight.

Take now the case of the settled heap, which, owing to the forces of gravity, impact due to rain, and other conditions with which we are not familiar, has been metamorphosed and is now a material which has decided qualities for withstanding compression: It will be found that a mass, 1 in. square and of a unit in length, if separated, say by a saw, and suspended as before, will not only develop an ultimate tensile strength high enough to carry its own weight, but possibly high enough to carry an additional load.

By these two cases the writer attempted to show that earth in

such a condition that it will assume an angle of repose is a far different material, from the standpoint of physical characteristics, than earth which has been subject to prolonged settlement and superimposed compressive forces. Mr. Paige.

It is the writer's opinion that an angle of repose is not applicable to a bank which has been formed by cutting a trench through well-settled material. In such a bank, it is not until the material has been stressed to its ultimate strength, thereby causing a granular condition, and consequent failure, that an angle of repose is evident. Until the ultimate strength is reached, the bank possesses all the qualities of an elastic body to a lesser or greater degree; but these qualities vary, owing to a great number of local conditions.

The writer thinks that a quality to resist shear is quite evident in the case of a bank being under-cut by stream action. Here the bank will stand until its own weight has become such that the ultimate tensile and shearing strength of the material has been reached, thereby causing failure along a vertical plane. The bank, in this case, falls intact, and becomes granular, due to impact alone.

Now to take up the manner in which a bank fails, the material of which is such that it will fail at a certain constant depth: Several years ago, the writer spent a summer with a field party of the United States Geological Survey, part of the time in a section where land-slide topography was common. At the time, his attention was attracted by the phenomenon spoken of by Mr. Haines in connection with small banks, namely, that the original surface of the ground was practically undisturbed in mass, though tipped at quite an angle inward, as shown in Figs. 17 and 60. At the time, the writer was surprised to note this condition, which was just the opposite of what might be expected, but he made no attempt to explain it until the case was again brought to his notice by reading Mr. Haines' able discussion.

Here there are two cases of land slide: on one hand, millions of cubic yards of material, consisting of heavy alternating beds of rock and soft shale; on the other, a bank, possibly 20 ft. high, and consisting of homogeneous material. Yet both act in very much the same manner, showing quite plainly that it is due to a general law of Nature, as might be expected.

A few years ago, the writer, while connected with the engineering force of a railroad during construction, noted that during the excavation of certain borrow-pits where cave-ins occurred and men were caught in them, that the men were never hit on the head by masses of earth falling from above, but were nearly always caught around the feet and legs, and thus tripped and overcome. This condition tends to show that failure in an unbraced bank, carried to a depth where it will fail for the first time, occurs somewhere near the bottom, as

Mr. Paige might be expected, for, in the writer's opinion, the bank fails because its own weight becomes so great that the ultimate strength of the material is reached. The greater the depth, the greater the dead load, and therefore it is natural to expect the larger stress to occur at the bottom.

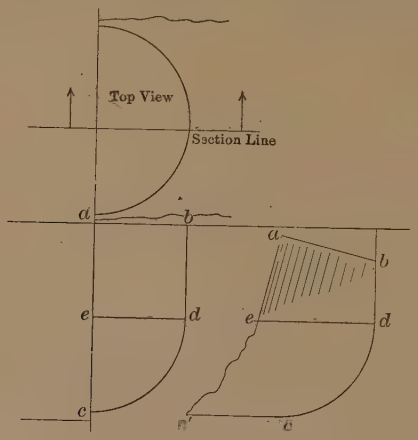


FIG. 59. FIG. 60.

The writer agrees with Mr. Haines that the tendency of a bank to fracture along the curved surface, $c - d$, Fig. 59, is actual, but he does not believe that this surface should be extended to a half circle, or that the only reason for fracture along the line, $b - d$, Fig. 59, is that it is a line of least resistance.

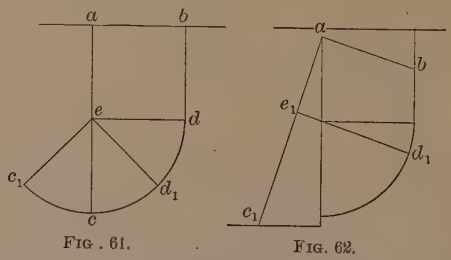


FIG. 61. FIG. 62.

Now, as to the causes which are responsible for the failure of a bank, as shown in Fig. 60: Assuming a regular arc, for simplicity, and that the movement along the arc, $c - d$, Fig. 61, is frictionless, it is known that the wedge-shaped area, $c - e - d$, Fig. 59, will assume such a position as shown at $c - e - d$, Fig. 61. Assuming, also, that the distance, $c - a$, Fig. 59, is the constant depth at which a certain material will fail for the first time, it is evident that the movement of the wedge, $c - e - d$, Fig. 59, in failing, must be of an inward tipping character, the final position of the wedge, just before becoming granular, being somewhat as shown in Fig. 62.

Mr. Paige.

A movement of this kind will account for three conditions which are noted in actual failure in the field:

(1) The inward tipping of the original surface, $a - b$, Fig. 60;
(2) The fact that a certain portion of the block, $e - a - b - d$, Fig. 60, is not disturbed in mass, showing that it has not been subject to either stress or impact during failure of the bank;
(3) That a bank does, and has been seen to, fail in the field at a point somewhere between c and e, Fig. 59.

The conclusions that the writer draws, from the action of this wedge, are as follows:

(1) That the failure of the wedge is due to certain shearing stresses acting along the arc, $c - d$, Fig. 59, of such magnitude that the ultimate strength of the material has been reached;
(2) That a bank carried to the known depth of first failure of a certain material can be retained as well by the bracing of the bottom half as by the top half, with the exception of the inconvenience due to the effects of erosion on the top edge of the bank and the consequent fall of small masses of earth on the workmen below;
(3) That, if such a bank is braced over its upper half, the pressure on the wedge, $c - e - d$, Fig. 63, will be relieved, owing to a horizontal arching action between the bracing and the stable portion of bank, and that the wedge, $c - e - d$, Fig. 63, will be retained in position at least for a certain time, depending on local conditions due to friction, along the arc, $c - d$, Fig. 63, and tensile strength of material, which latter term, as used by the writer, includes many conditions of molecular adherence of which nothing is known at present.

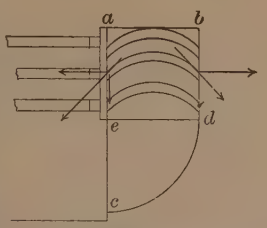


FIG. 63.

In applying the term "horizontal arching action," the writer wishes to call attention to a condition which he thinks is of the utmost importance in the study of earth pressures. Take, for example, a bed of rock which is overlaid by two or three miles of solid rock: It is known that, per unit of area, this bed of rock does not receive a load in direct ratio to the dead weight of a column of rock of a section equal to the unit area. It is known that some type of arching action relieves the stress on the lower strata, just as the arching action would

Mr. Paige. in the case of a hollow sphere subject to an externally and uniformly distributed load, the tangential compressive stresses acting in the shell of the sphere much as the forces of gravity cause lateral compressive stress in the crust of the earth.

Mr. Haines, in Fig. 31, applies an arch which he stands on end. He states that, as long as the ultimate strength in compression at *D* is not exceeded, the arch will stand; in other words, that it will be in equilibrium. The writer's opinion is that this arch would not stand at all if it approached so near to the point, *D*, that the dead load of the earth from *D* downward to the arch is not enough to balance the vertical component of the upwardly inclined reaction which is common to all arches if placed in such a position as shown. If this arch did go to a point very close to *D*, as shown in Fig. 31, then there is no weight of material to overcome this component, which has a tendency to shear the material along a plane normal to the compressive forces acting at the foot of the arch, or *D*, Fig. 31.

In the writer's opinion, an arching action such as shown in Fig. 63 will prove worthy of investigation. Here there is an arch, the thrust of which is taken up by the bracing on one side and the stable portion of the bank on the other, while the vertical component of the oblique reaction is taken up by the friction on the bracing and the stable bank, thus relieving the wedge, $c - e - d$, Fig. 63, from the effect of the dead weight of the mass, $e - a - b - d$. As stated before, the wedge, $c - e - d$, remains in place because of adherence and friction along the arc, $c - d$.

The foregoing discussion has dealt with the distance, $c - a$, Fig. 59, assumed to be the depth at which a certain material will fail for the first time. The writer will now attempt to show how the same material, with its same constant depth of failure, will act when this trench is carried still deeper. Referring to Fig. 64: Bounded by $R - A - B - 1$, the first cut is carried to the depth of 10 ft., which depth will be assumed to be its constant depth of failure.

Now, carrying the trench 5 ft. deeper, to a total depth of 15 ft., then, by following such approximate fracture planes as occur in the failure of a bank in the field, it is found that the distance, $A - B$, is equal to half the constant depth of failure, and that the distance, $2 - C$, is only $7\frac{1}{2}$ ft., so that, at a 15-ft. depth of trench, one may expect that portion of the bank back of $2 - C$ to be in a stable condition. Carrying the trench 5 ft. deeper, a total depth of 20 ft., it is found that the distance, $3 - D$, becomes 10 ft., and that one may expect the material to fail at this point in the same manner as it did at the point of first failure. Two points of interest may now be observed in the diagram:

(1). That the second point of failure in the bank is not reached until the depth of the trench is twice the depth of constant failure;

(2).—That the center of gravity of the wedge, 3 — 10 — 18, coincides with that of the first wedge, Q — R — 1, also that a line passed through the centers of gravity of these two wedges is just a trifle more than one-third of the height of the trench from the top. Mr. Paige.

From the diagram, it is now seen that each increase in depth of trench beyond 20 ft. will cause additional failure for the line, 3 — D,

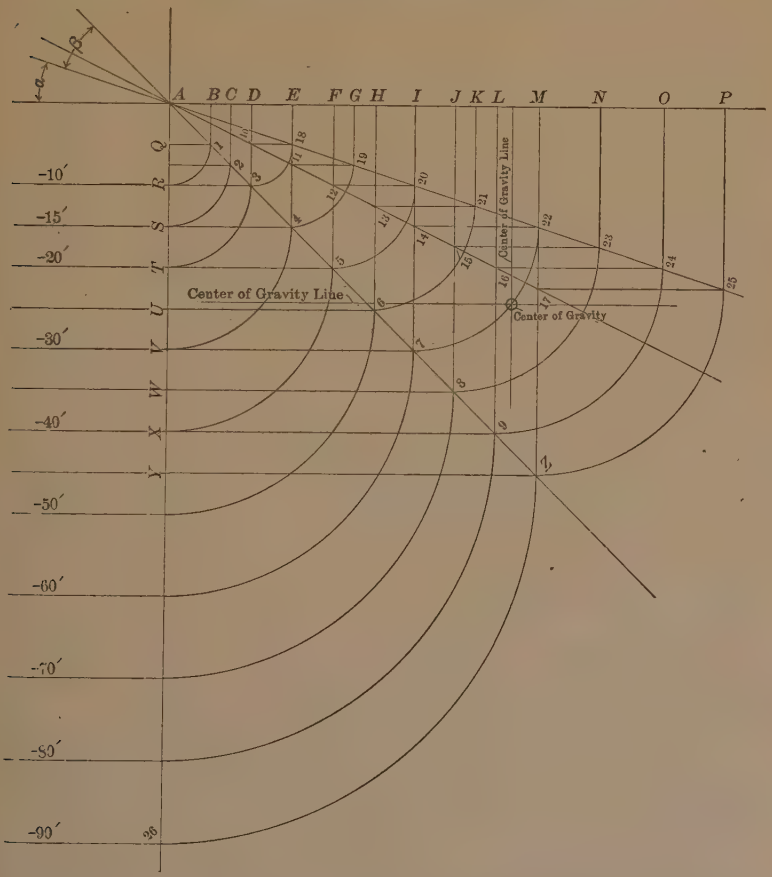


FIG. 64.

increased in length for each increase in depth. Carrying the trench first to 30 ft. and then by 10-ft. stages to a depth of 90 ft., and constructing the respective wedges of failure, it is found that the diagram has developed several new points of interest:

(1).—It is seen that a line drawn through the points, 1 — 2 — 3 — 4 to Z, will pass through A, and will form an angle of 45°, both

Mr. Paige. with the surface of the ground and the face of the trench. This condition was to be expected, and is due to the fact that the radius of the arc, R — 1, in the actual failure of a bank in the field, is found to be equal to half the depth at which the material will fail.

(2).—That a line through the points, 18 to 25, inclusive, if extended, will also pass through A, and that, from 18 to 25, it is a locus of the right hand upper point of all the possible wedges of failure formed after twice the constant depth of failure has been reached and exceeded.

(3).—That, taking only the three wedge areas, 3 — 10 — 18, 4 — 11 — 19, and 5 — 12 — 20, the center of gravity of these combined areas falls at a distance from the top a trifle less than one-third of the total depth of the trench.

(4).—That a line passing through the respective centers of gravity of the triangles, A — T — 5, A — V — 7, and A — Y — Z, will fall at a point distant from the top equal to one-third of the depth of the trench which develops any of the above mentioned triangles.

(5).—That, by approximate calculation, it may be seen that the area, Z — 3 — 10 — 18 — 24, increases more than twice as fast as the depth of trench, and also that the center of gravity of the continually increasing wedge area falls at a point about 28% of the total depth of trench from the top.

(6).—That a line passed vertically through the center of gravity of the triangle, 25 — A — P, will connect with a line passing horizontally through the center of gravity of the triangle, A — Y — Z, by a quarter circle terminating on the lines, A — Z, and A — 25; also, that the center of gravity of the wedge area is at the intersection of this quarter circle with a line passing horizontally through the center of gravity of the wedge area.

(7).—That tan. $\alpha = \frac{1}{8}$, and tan. $\beta = \frac{1}{2}$.

Mr. Haines has mentioned the fact that in the field he had noted no fracture of the bank back of the first which appeared on the surface, even though the trench were carried considerably deeper. The writer would account for this in two ways:

(1).—That the fracture zone was possibly one caused by a give in the bracing, due to new wedge areas coming into action, and that it might not be of necessity a fracture caused by arrival at the point of first failure;

(2).—That material in a bank which has a constant depth of failure is in such a condition that it does not have to fracture in order to exert pressure. Before the excavation of a trench, the material is under action of compressive forces which tend to reduce its volume; and, the instant a trench is dug, lateral compressive forces are removed in one direction and the material has a decided tendency to increase in volume, thereby exerting pressure on any plane which

opposes such expansion. On this assumption, the bank in the diagram would not have to fracture along the lines, $D - 3$, $6 - H$, or $8 - J$, in order to cause pressure on the face of the trench wall, $A - Y$, but, if the bracing restraining the trench wall were to give, such a fracture might occur. Mr Paige.

This diagram, the geometrical features of which are developed from the manner in which banks or land slides actually fail in the field, is interesting principally from the fact that it shows quite clearly that the greatest stress in the bracing restraining a bank may be looked for at a point about one-third down from the top. It would also lead one to believe that the stress varied directly as the wedge area, but until one knows what distribution of load and stress is the cause of the curved fracture zone, it is of no value in calculating such pressure. It is the writer's opinion that, even if he is at some future writing able to show mathematically the cause of this curved fracture and the consequent increment of horizontal stress developed in the wedge, it is doubtful whether such information would be of value until a great many experiments had been made tending to show the actual pressures exerted by material as in a bank, also experiments which will show the ultimate values, in compression, tension, and adherence, of most of the common classes of material excavated. These figures would then vary a great deal from those actually developed in the field, owing to local conditions, the presence of moisture and its percentage being paramount. It is the writer's opinion that the tendency toward a curved fracture zone, both horizontally and vertically, is caused by the varying density of the material, due to the removal of lateral compressive forces.

As to the reason for the wedge leaving the bank at a point about 6 to 12 in. above the actual bottom of the trench, as shown in Fig. 59, the writer thinks that this fact is ample proof of the elasticity of the material, for the shearing stresses must be carried to the undisturbed portion of the material at the bottom in order to cause equilibrium; and, further, in order that the material be able to do this, it must develop elastic qualities.

The writer now desires to quote from the discussion by R. B. Stanton, M. Am. Soc. C. E., on the paper entitled "Lateral Earth-Pressure and Related Phenomena,"* in which Mr. Stanton describes briefly two large land slides on the Thompson River, in British Columbia. Those portions of the discussion of the most interest to the writer are as follows:

"At both places the country orginally sloped up from the river in a series of benches or terraces to the first line of hills. The south slide has an extreme length of 1 880 ft. along the railway, and an extreme width, back from the river, of 1 575 ft. It is of somewhat

* *Transactions*, Am. Soc. C. E., Vol. LIII.

Mr. Paige. irregular form, with a semicircular outline at the back, and covers an area of 66 acres. The north slide has a maximum width at its widest portion of nearly 0.5 mile, and a length, back from the river, approaching 0.75 mile, with the same semicircular back line. It is of irregular form, and covers an area of 155 acres. The height of the first bench next to the river, in both cases, was originally about 80 ft. above low-water level. The land then rose in successive levels to a height, on the south slide, of 400 ft. to the bench at the top, or back edge, where the cave-down broke off the solid ground, and, in the case of the north slide, it extended to the third higher bench 500 ft. above the river. It is impossible to ascertain at what depth these enormous masses of earth and loose rock broke, or, in other words, the depth of the plane on which the mass moved toward the river; but it is estimated that at the back edge of the south slide the break fell almost vertically for a distance of more than 300 ft., and on the north slide perhaps more than 400 ft.

"* * * in the case of the south slide, estimated as weighing some 32 000 000 tons, and of the north slide approaching 100 000 000 tons—the whole mass dropped vertically, while the immense tracts of broken and mixed material seeking an outlet forced their lower sides out on the line of least resistance and found their way into the river. This action is distinctly shown by the almost vertical walls in the boulder clay along the outline of these two slides. While at their foot there is now a talus slope of crumbled material, these walls stand vertically to a height of from 50 to 200 ft., more clearly shown on the north slide, where the vertical cliffs of boulder clay, and in places the silt itself, extend around the whole slide for a distance of more than 1.5 miles. It is also shown by the present position of large sections of the original surface of the highest bench, which broke off at the line of the back wall, and which now stand in the sunken mass at an angle of about 45°, with their former level surfaces tilted back and away from the river. The back edge thus dropped first and lower than the portion some distance in front of it. In dropping and pushing out toward the river, the whole tract was broken into sections by great cracks, which still exist. The larger cracks run parallel with the river and at right angles to the line of movement, while the other and smaller cracks run in every direction, cutting the whole into blocks of boulder clay and dry silt."

Of the many interesting points in this discussion by Mr. Stanton, the most interesting proved to be the existence of these large cracks which run normal to the direction of movement of the slide. Such cracks would be due to planes of movement such as shown in Fig. 64, at $F - 5$, $I - 7$, or $J - 8$.

In Fig. 64, considering all material below $Y - Z$, it is the writer's opinion that the wedge below $Y - Z$ will be retained in position in the event of the upper half of the trench being adequately braced, owing to the same forces which retain the far smaller wedge in the case of first failure of the bank, that is, friction along the arc, $26 - Z$, the relief of the load, $Y - Z$, due to horizontal arching action and the adherence or tensile value of the material.

Mr. Paige.

With reference to driving sheet-piling: At a certain point in the driving, it is noted that a sudden increase in resistance occurs. This may be explained in the following manner, referring to Fig. 65: In driving a pile, the material within a certain distance is stressed beyond its ultimate strength, thereby becoming granular, and also, owing to the lateral compressive force exerted on the material by driving the pile, it follows a line of least resistance, that is, upward. As soon as the pile reaches the constant depth of failure of the material, there is a tendency for the material to fail, as is the case in Fig. 59, action being from all sides and the pressure on the pile greatest at or near its bottom. This failure of the stable portion of the material the writer thinks is due to the reduction of lateral compressive forces, owing to the rising of the material around the pile, as shown in Fig. 65. This condition has been noted by the writer quite frequently in driving round piles and also sheet-piling.

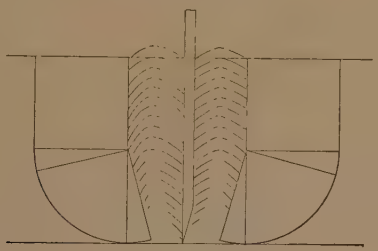


FIG. 65.

As to a retaining wall, it is the writer's opinion that, at the time of back-filling, an angle of repose can be applied in calculating the forces to which the wall is subject; but that, after a few years, or when the material has become well packed, the wall is stressed in a far different manner than at the time of back-filling.

Finally, it is repeated, that the diagram, Fig. 64, is based only on the action of a material which will stand alone to a certain constant depth when excavated, and this reasoning will not necessarily apply to material which contains a large percentage of moisture, such as "swelling ground." Such a material as will stand alone must develop the characteristics of an elastic body in order to do so, that is, ultimate tensile, compressive, and shearing unit stresses. Any land slide or bank that fails, and in so doing develops the inward tipped original surface together with a nearly vertical fracture from the stable ground, cannot have moved along a plane the cross-section of which will be a right line, but, of necessity, must have moved outward and downward along a curved plane, not necessarily a quarter circle as assumed for simplicity in Fig. 64, but any curve, such as a parabola or hyperbola. The percentage of moisture in a material has more to do with the

Mr. Paige. manner in which it will act than any other single cause, in the cases of loams, clays, sands, gravels, etc.

The writer believes that what is needed most, in lateral earth pressure investigation, is a general law which is known to be true for material of an average class. Until such a law is developed and proven to be true, it will not pay to attempt to derive formulas for calculating the pressure on bracing. Such a general law, if developed, will have to be supplemented by the results of many tests and experiments made on actual banks in the field, before it can be applied in formulas.

The writer believes that, in nearly all cases, it is economy to avoid failure, with its consequent delays and excessive unit cost for clearing and reconstruction, and when such successful and capable men as Messrs. Meem, Haines and Goodrich vary so much in their methods and results in calculating stresses in bracing, it is quite evident that such trench bracing as has done its work is far more likely to have been excessive than economical.

At the present time, Fig. 64 is submitted only with the hope that, if deserving of investigation by the Profession, such investigation will be a step toward the development of a new general law for the movement of earthwork in some of the common problems with which the engineers of to-day are confronted.

Mr. Gifford. L. R. Gifford, Assoc. M. Am. Soc. C. E. (by letter).—This paper relates to a subject on which very little reliable information is available. There are several reasons for this lack of data, and possibly the principal one is the fact that the pressures exerted by earth are difficult to determine experimentally, and vary within wide limits for the same material under different conditions of moisture. The variation in the moisture, moreover, may take place within very short periods of time, causing the material to change from dry to saturated by reason of heavy rainfalls.

The author, as the result of his observations, proposes a new theory of earth pressures, to be used in designing sheeting and sheet-piling. This theory reverses the usual assumptions, and places the resultant of the pressure at one-third the depth from the surface, instead of at two-thirds, as is customary.

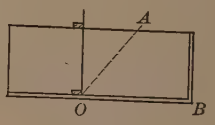


Fig. 66.

In order to test this theory, for a general case of a dry granular material, the following experiment was made with a box 5¼ in. deep, about 15 in. long, and 7½ in. wide, one end being replaced by a piece of stiff cardboard arranged to act as a simple supported beam. (Fig. 66.)

The length of the box was such that the angle of repose, *A O B*, of the material, which was a fine granular sand made of crushed blast-furnace cinder, would fall well inside the box. The angle, as deter-

mined by actual measurement, was about 35 degrees. In filling the box, care was taken to place the sand in thin horizontal layers. Mr. Gifford.

The deflection of the cardboard side, due to the pressure developed, is shown in Fig. 67, which is typical of several experiments.

Curve 1 is the position assumed before any settlement was noticeable on the surface.

Curves 2, 3, and 4 are the successive positions assumed after the surface had settled perceptibly, due to the deflection of the cardboard, caused by a slight jarring of the box.

It will be noted that the point of maximum deflection is slightly below the center, as was the case in all the experiments made.

Considering now the case of a beam with a uniformly varying load, as shown in Fig. 68, it follows that the theoretical point of maximum deflection is 0.51 l from the support, A, which agrees very well with the result secured in the experiment.

Considering the assumptions of the sliding wedge of earth as made by Mr. Meem in Fig. 6, it would follow, using the same line of reasoning, that similar triangular elements should be considered in computing the pressure on the rangers and braces, and not the horizontal elements, as he has done in Fig. 7.

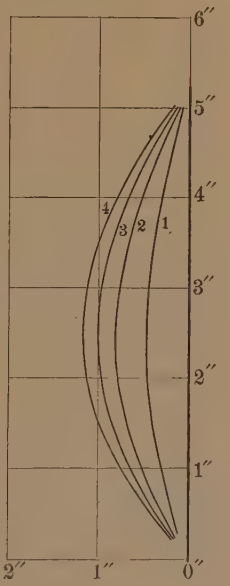


Fig. 67.

As pointed out by Mr. E. F. Jonson, this assumption of triangular elements leads to the conclusion that the greatest pressure is at the bottom of the trench, and that the point of application of the resultant pressure is one-third of the depth above the bottom.

The foregoing discussion by the writer must not be considered as in any sense an argument tending to disprove the existence, under certain conditions, of a greater pressure on the upper portion of sheeting supporting trenches and the like. It seems fair, however, to assume that, with granular material, as in the case of true hydraulic pressure, the horizontal pressure increases directly with the depth, and, accepting the author's observations of a decrease of pressure with the depth, it follows that there may be any intermediate condition between these two extremes.

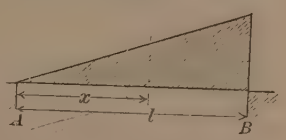


Fig. 68.

With the present knowledge of this subject of earth pressures, it seems to be practically impossible to calculate in advance, with any

Mr. Gifford. degree of certainty, the pressure for which provision will have to be made in any given piece of work, and this naturally leads to a lack of economy in the sheeting and bracing.

The writer has recently become much interested in the subject of steel sheeting and sheet-piling, and, from the studies he has made, he believes that it is possible to secure a steel sheeting which will overcome many of the difficulties attending the use of wood sheeting, and at the same time provide a more satisfactory and economical construction, the question of the economical design of the sheeting being naturally the result sought from a study of the pressures.

There are in general use, at the present time, several forms of steel construction suitable for covering surfaces subjected to pressure:

1st.—Plates.

2d.—Buckle-plates.

3d.—Corrugated iron.

4th.—Trough construction of plates and angles as in bridge floors.

5th.—Steel piling, of structural shapes or special rolled sections.

Of these forms, the first and second, if used for sheeting, would require stiffening members, the fourth would provide excessive strength, except possibly in very unusual cases, and the fifth is not suitable for ordinary trench work.

The third, while too light for use as sheeting in the sizes now manufactured, seems to offer a very satisfactory form, if properly designed and of sufficient thickness and depth of corrugation.

The writer submits the following analytical determination of the approximate necessary dimensions of a corrugated plate in order that it may be equal in strength to timber sheeting of the usual thickness, which may be taken as 2, 3, and 4 in.

Assuming as a basis of comparison the coefficient of strength, that is, the safe load uniformly distributed on a span of 1 ft., and for convenience, considering also a width of 1 ft., the results for timber are:

W = total load, in pounds, that is, the coefficient of strength;
S = section modulus;
f = fiber stress allowed in bending = 800 lb. per sq. in;
t = thickness of timber, in inches;
l = span, in inches = 12.

$$S = \frac{12 \times t^2}{6}$$

$$\frac{Wl}{8} = fS$$

$$W = \frac{f \times S \times 8}{l}$$

Therefore $$W = \frac{800 \times 12 \times t^2 \times 8}{12 \times 6} = 1066\tfrac{2}{3}\, t^2$$

Mr. Gifford.

For 2-in. plank, $W = 1\,066\frac{2}{3} \times 4 = 4\,266\frac{2}{3}$.
3 " " $W = 1\,066\frac{2}{3} \times 9 = 9\,600$.
4 " " $W = 1\,066\frac{2}{3} \times 16 = 17\,066\frac{2}{3}$.

The dimensions to be given to the corrugated iron can now be considered.

Assuming that, for the corrugated section, the coefficient of strength is given with sufficient accuracy by the formula:

$$W = \frac{37\,500\, h\, t^*}{L^2}$$

Where W = safe load, in pounds;
L = span, in feet;
h = depth of corrugation, in inches;
t = thickness of metal, in inches.

The fiber stress is assumed to be 15 000 lb. per sq. in., and is included in the constant, 37 500.

Taking the span at 1 ft., and the depth of corrugations at 2 in., and $2\frac{1}{2}$ in., it follows, for $h = 2$ in., that:

$$t = \frac{W}{37\,500 \times 2}$$

and for $h = 2\frac{1}{2}$ in., $t = \frac{W}{37\,500 \times 2\frac{1}{2}}$

Therefore, for the equivalent of 2 in. timber, for $h = 2$ in.,

$$t = \frac{4\,266\frac{2}{3}}{75\,000} = 0.057, \text{ or } \tfrac{1}{16} \text{ in.}$$

Therefore, for the equivalent of 3 in. timber, for $h = 2$ in.,

$$t = \frac{9\,600}{75\,000} = 0.128, \text{ or } \tfrac{1}{8} \text{ in.}$$

Therefore, for the equivalent of 4 in. timber, for $h = 2\frac{1}{2}$ in.,

$$t = \frac{17\,066\frac{2}{3}}{93\,750} = 0.180, \text{ or } \tfrac{3}{16} \text{ in.}$$

For work of the temporary character of sheeting, it would probably be entirely practicable to use a fiber stress of 20 000 lb., and it should also be noted that the closer the corrugations approach a semicircular form, the more economical the section becomes as regards the ratio of weight to strength, and, for this reason, it might be advisable to use a more nearly semicircular form than is at present used for corrugated iron.

It will be seen that these sections are of such dimensions that the sheets could be quite as easily handled and driven as the timber, and could readily be made, say 18 or 20 in. wide, which would reduce the number of pieces to be handled.

There seem to be good reasons for believing that steel sheeting,

* For the derivation of this formula, see *Engineering News*, Vol. 58, December 12th, 1907, a communication from George H. Blakeley on "The Strength of Corrugated Sheeting."

Mr. Gifford. such as here proposed, would prove more satisfactory than timber sheeting, and among these may be mentioned the following:

1st.—A comparative estimate, of the cost of timber piling and a corrugated-steel piling of approximately the dimensions calculated above, indicates that a decided saving can be secured by the use of such steel sheeting as here proposed.

2d.—The withdrawal, after back-filling the trench, will not cause settlement, as is the case with timber, on account of the smaller bulk of the steel. This is a particularly important feature.

3d.—By lapping one corrugation in driving, a sand and water-tight job can be secured.

4th.—There would be no split piles, as with timber.

5th.—It would be easier to drive and withdraw than timber.

6th.—It would be so much more durable than timber that it would be practicable to use it at least ten times, with an ultimate salvage value as scrap steel. Timber can be used only once, or twice at most, by recutting, and has little or no salvage value.

7th.—The greater strength which can be given to the steel sheeting will result not only in a saving in the quantity of rangers and bracing, but—which is fully as important—the working space can be made larger and will be less encumbered.

Mr. Meem. J. C. Meem, M. Am. Soc. C. E. (by letter).—Before concluding this discussion, it is necessary to show that there has been a misconception, on the part of some of the discussors, as to the differentiation which was made in the paper between earth pressure and aqueous pressure. The paper sets forth clearly the writer's interpretation of the difference in the actions of these well-defined types of pressures, but, in order that there may be no further misunderstanding concerning them, the materials, against the pressure of which the engineer has to provide, will be divided into three classes:

Class A ("Hard Ground").—Solid rock, hardpan and hard, dry clay, or, in general, any material which will stand indefinitely with a vertical face unsupported.

Class B ("Soft Ground").—Ordinary clay, loam, dry and moist sands, gravel, broken stone, or, in general, any material which will assume and maintain a definite angle of repose when allowed to fall freely on a platform.

Class C ("Aqueous Material").—Liquids, fluid earths, such as pure quicksand, viscous material, such as "bull liver" or asphalt, or, in general, any material which will not assume or maintain a definite angle of repose, but which, eventually, will seek its level.

To avoid further misunderstanding, it is stated that all submerged or subaqueous material should be included with Class C, with the re-

strictions to be noted later. The writer asks to be pardoned for defining further: that the terms, "natural slope" or "slope line," is used herein by him in what he understands to be the generally accepted sense, that is, the slope which material, when dry, will assume if allowed to fall freely in a pile, the angle between this line or plane and the horizontal being the angle of repose. Mr. Meem.

In addition to these three classes, there are a few kinds which defy classification, such as clays which swell and ooze through sheeting, and for which openings have to be left, and clays permeated by veins of quicksand, which are as apt to slide horizontally as in any other direction.

It is generally conceded that the materials noted in Class A do not require any bracing, except occasionally to retain detached fragments in place, and it is also conceded that materials of this class arch themselves. Again, it is beyond debate, and is clearly set forth in the paper, that the materials in Class C exert hydrostatic pressure, and that these pressures are covered by the universally accepted formulas therefor. In reference to this matter, the writer may say, parenthetically, that he does not share with one of the speakers his fear that a careless reading of this paper may cause some young engineer to design a structure which may fail under hydrostatic pressure. He trusts he may be pardoned for his candor when he says that his fear heretofore has been that some engineer, taking too literally the accepted theories of earth pressures in ordinary ground, may design bracing or other structures which may fail. The paper states, concerning aqueous pressures, that the full value of the hydrostatic head should always be taken in calculating pressures in subaqueous structures, although it is admitted that pressures corresponding to its full value do not always obtain. Where, for instance, any porous earth, such as sand, is submerged, the condition may be illustrated by comparing it to an indefinite number of tubes through solid material, each filled with water and opening against the face of a structure, the tubes, of course, corresponding to the voids in the earth. The effect of this is to give a full pressure (as would be shown by a gauge) at the mouth of each tube, but a diminished pressure over the whole area corresponding to the percentage of solid matter pressing directly against the structure. There is, however, a corresponding percentage of pressure from the earth or solid material, which is measured by a diminished tendency to slide along the angle of repose, and therefore to arch itself against the upper sheeting, giving in some instances, as in coffer-dams, a somewhat greater pressure over the whole area than would be the case for water alone. In shafts and enclosed structures, however, this pressure is diminished in all cases by the arching effect of the solid material, except where the cover is so light as to preclude the possibility of its arching. It is often noted

Mr. Meem

that, in building subaqueous structures, the heaviest pressures, at times causing deformation, occur during or shortly after the progress of the shield. This is due to the semi-liquid condition in which the out-leak of the air and the in-leak of the water leaves the earth, and, as soon as the leaks are stopped and the material settles down and compacts itself, its arching tendency is brought into play and relieves the structure of much, and in some rare instances of all, of the hydrostatic pressure. The exigencies of construction, however, demand that the full hydrostatic head, as stated in the paper, shall be considered in designing, it being well to note that this pressure is based on the specific gravity of the liquid, and not its weight per unit together with that of its suspended matter.

It is safe to say, then, that any submerged structure, which is able to withstand the operation of being built, has in reserve a large factor of safety governing its future stability.

This does not apply to tunnels in soft ground built by methods which allow an excess of voids to remain above the roof, or which leave large timbers in place to rot out and cause slumps in the superimposed material. The effect of friction, in diminishing the hydrostatic pressure over the whole area of a structure in submerged earth, is shown by the fact that, in sinking caissons rapidly, and sometimes in building tunnels, it is not always necessary to use air pressure corresponding to the full hydrostatic head. It is also shown indirectly by the fact that internal water-proofing can be and has been applied successfully to masonry which was otherwise porous.

As to the question of water getting behind sheeting and causing hydrostatic pressure, it is difficult to conceive of such a condition, except in a coffer-dam or a trench in the bed of a stream, subject to floods and droughts, or similar conditions under which pressures should be calculated for the full hydrostatic head. In ordinary sheeting, provision should be made to divert a heavy flow of water, in cases of flood. In small quantities, water may percolate or seep through the sheeting, as it does through the ground, without doing any harm. When large quantities of water rush through spaces in the sheeting, the bracing fails, not because of the hydrostatic pressure in itself, but because of the hydraulic mining, taking place behind the sheeting, which destroys the value of the bracing, and causes the sheeting to loosen and drop out.

The writer has noted two specific cases in which tunnels, under his supervision, in sand, one for a sewer 4 ft. in diameter and one for a sewer 14 ft. in diameter, were entirely flooded, and also several cases in which open cuts, both large and small, were badly flooded—all absolutely without damage, either in the act of flooding or the operation of unwatering.

In all these instances, the sheeting and bracing were known to be

tight, and there was no opportunity for the hydraulic mining which was witnessed in another trench in which a large sewer broke behind the sheeting during a flood. The bracing collapsed, not from overstrain, but because it lost its support from behind. Mr. Meem.

Now, as to the arching effect of "soft ground," or the Class B materials: Anyone who has delved in the ground at all must be aware of numberless instances of the arching effect of this material which have come under his notice. That some kinds of ground of this class do arch themselves is generally admitted. It is the admission, however, of the fact as a general and accepted theory that seems to be objected to by many engineers. It has been the purpose of this paper to show:

(*a*).—That all materials of Class B arch themselves, and do so in accordance with a fixed physical law, which finds expression in a theoretical formula;

(*b*).—And that this arching effect is transmitted to bracing or to the face of a wall which bisects its key and takes the place of one-half of the arch, and that it is governed by the same general law and may also be expressed by a formula.

If it is true that "soft ground" does not arch itself, then it would be as impracticable, for instance, at a depth of 300 ft., to brace a narrow tunnel having a width of 20 ft. as a broad one having a width of 200 ft., and this, of course, does not admit of argument.

Numerous instances have been given of tests of concrete slabs and other beams in which the supported material arched itself, and made the test of no value. For instance, in Fig. 70, in testing a slab, with the material piled as shown, this slab is not stressed by the full weight of the material vertically above it, and if the material is piled higher, or to an indefinite height, it will not make any real difference in the carrying capacity of the slab. It will not be disputed, the writer thinks, that the slab sustains only a certain percentage of the weight of the material, above which the main body of earth forms itself into a true arch, bridging the space between the piers and being capable of sustaining not only its own, but additional weight, in proportion to the thickness and strength of the key.

Now, suppose that one-half of this material is removed and that sheeting and bracing is substituted therefor, as shown by *G H*. Is there any reason to believe that the stresses are changed because bracing has been substituted for what was formerly the key of an earth arch? The slab is still carrying the weight of a portion of the loose earth below, and the remainder is arching itself somewhere between the bracing and the slope line, *x y*. The writer believes that this test applies to dryer sand as well as to moist sand and loams, except, of

Mr. Meem. course, that the dryer the sand the less its cohesion, and therefore the greater the load on the supporting slab for the same thickness of key.

Again, in Fig. 69, the tendency of a strut, *C O*, is to exert pressure at *O* against the face, *A O*, in the proportion of *J K* to *J L*. The tendency of an indefinite number of struts piled one above the other to slide along the plane, *C O*, could be resisted by the interposition of a plane, *J P*, at right angles to the line of these struts, and this plane, *J P*, would exert pressure against *A O* in the proportion, *P L* to *L J*. Assuming the case of a flat angle of repose, *L J* would be approximately half of *P L*, which is the writer's reason for bisecting the angle between the vertical and the angle of repose and assuming that the weight of the entire mass, *A O B*, acts as direct thrust against the line, *A O*. It is probable, however, that this material arches itself along the lines, *N H*, *P G*, etc., and the greater pressure, due to the

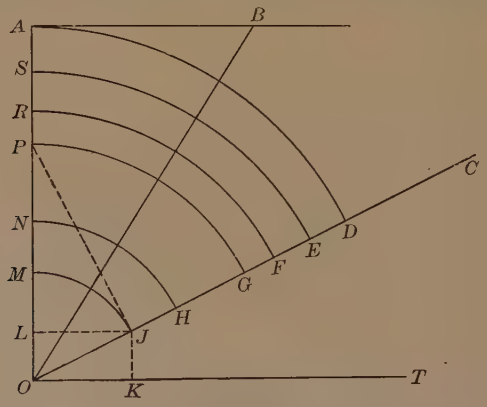


FIG. 69.

rise of the arch being proportionately shorter than the tangent of the angle, *L J P*, can be offset by reason of the fact that the friction of the mass prevents the full pressure from being exerted by holding the particles more securely in place, and thus, for want of more specific data on the subject, it may still be assumed that the thrust against *A O* is equal to half of the weight of the mass, *A O C*. Again, it may be assumed that *M J*, *P G* and *A D* are brick arches, built of dry brick, above, and independent of, each other. No one can gainsay the fact that each or all of the lower arches could then be removed without disturbing the stability of those above, and that the pressures against the face, *A O*, would be directly proportionate to the length and weight of the arches, and therefore greatest at the top. It is not difficult to follow this idea from dry brick to rubble, to loose rock, to gravel, to moist sand, and finally to dry sand, for there is no more physical reason why dry sand, under proper conditions, should not arch

itself than loose rock, it being understood, of course, that the lower loose particles under the imaginary intrados of the point of beginning of the first arch are not included in the arching effect which takes place above. It is well to note here that this arch or wedge partakes somewhat of the nature of a coherent solid, not because of its internal cohesion, but because of the external pressure, which gives it coherence, as long as this pressure is maintained, either by the bracing being put in tight originally, or by the wedging it gives itself by slipping down and adjustng itself to a solid bearing. Any cohesive or added frictional properties which these particles may have by reason of moisture is a factor of safety for the designer, and should not be considered except in doing rapid work, where the ground will not have time to dry behind the sheeting. For instance, in building a subway through a sandy soil in a region subject to drought and in which the bracing would have to stand unprotected for a long period, it would be unwise to rely on the pressures being less than those set

Mr. Meem.

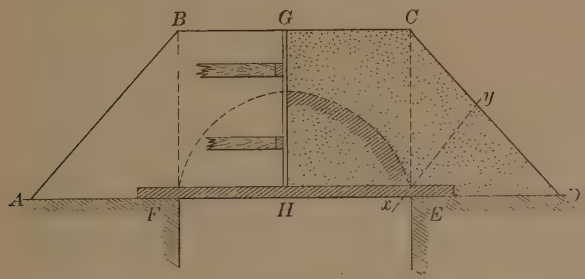


FIG 70.

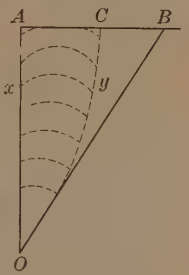


FIG. 71.

forth in the paper. This cohesive quality, by virtue of the ground being damp, may cause the material to arch itself somewhat as shown in Fig. 71, and the line, *C O*, may take the form of a parabola, as shown by Mr. Moran, or by Mr. Haines in Fig. 29.

The overhang shown in Fig. 40 is doubtless due to the protection afforded by the paving above and the greater compactness and cohesive qualities of the earth directly below, thus transforming into a cantilever what was previously a portion of an arch.

Plate VII is shown here to offset the criticism of one writer, who stated that the exposed area of sand shown by Fig. 1, Plate I, was too small to be of value in upholding the assumptions of this paper. This view shows the sheeted side of an under-cut tunnel braced with as light bracing as would be called for at the bottom of a trench 12 or 15 ft. deep. The horizontal sheeting, of 2-in. planking, here shown, is held in place by vertical 6-in. I-beams (on the flat) spaced about 7 ft. apart and engaged by the concrete footing below and the backs of the shoe-plates above. It is understood, of course, that the

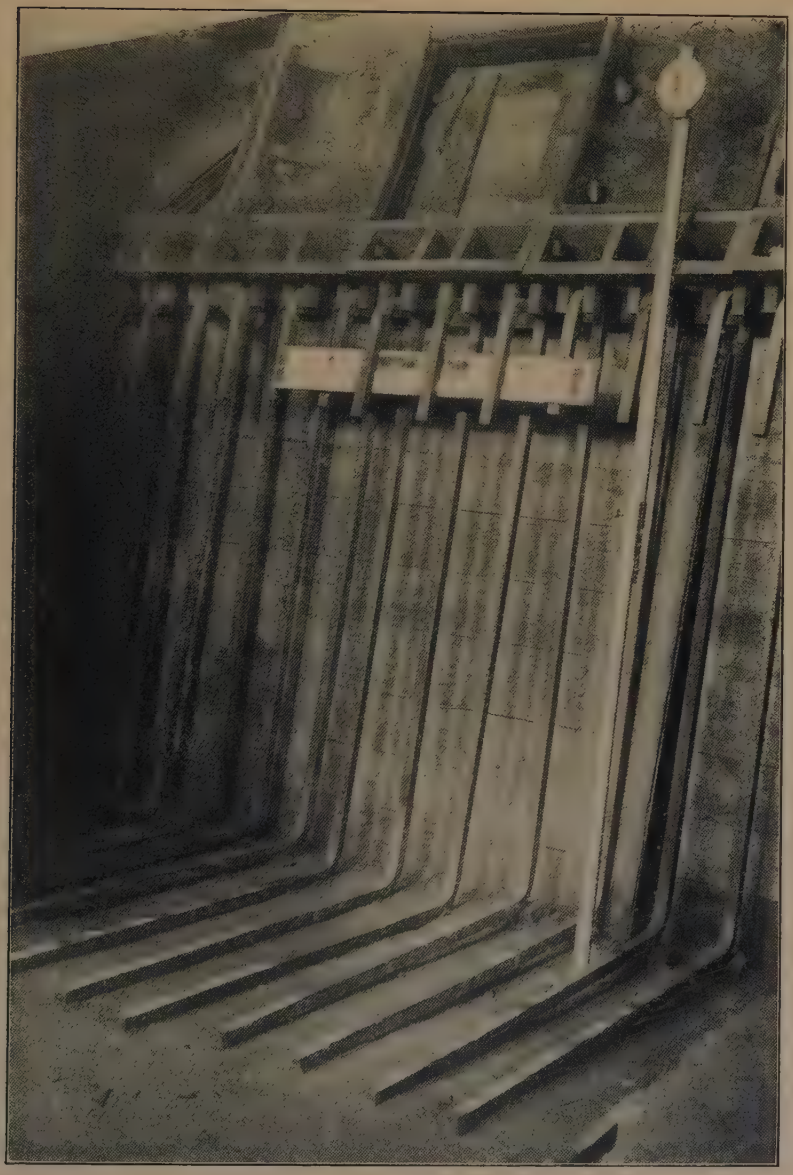
Mr. Meem. main body of the tunnel is carried on timber bents pending this operation. It is plainly evident that a face of sand of this area could not stand unsupported, and an analytical consideration of the subject should convince anyone that the sheeting is only necessary to hold in place the loose sand below the theoretical intrados, above which the arching effect of the material supports everything else in place. Sheeting put in as carefully and as tightly as this, without greatly disturbing the sand back of it, can be depended upon with absolute certainty as long as no violent disturbance takes place above or the excavation is not extended below this point.

Referring again to Mr. Haines' discussion, it appears to the writer that he and Mr. Haines are not very far apart, and it is of interest to note that, to a great extent, these discussions, prepared independently, follow the same line of thought. Mr. Haines will doubtless admit that, in absolutely dry material, the curve, *C O*, Fig. 71, would approach more nearly, if not quite, to the straight line, *B O*.

He also states that, through an oversight, the writer eliminates friction in the consideration of his earth pressure formulas. It is hardly necessary to say that the writer considers friction a very large element in his calculations, but for safe practice he has eliminated cohesion, on which Mr. Haines must rely for the results he obtains. Further, in deriving the theoretical straight-line formula, the writer, in order to apply a factor of safety to his formula, has considered the mass, *A O C*, Fig. 69, as tending to slide along the plane, *O C*.

In other words, the writer has considered the material as tending to act in the manner in which it would if the bracing, *A O*, were entirely removed and the mass were absolutely dry. He recognizes the fact that this is a condition which will be found in practice in the rarest instances, but he realizes that a large factor of safety, approaching an unusual but possible condition, is not undesirable in a formula of this character. It is very unfortunate that the experiments to which Mr. Haines refers, showing that the heaviest pressure was found near the top, were not carried to their logical conclusion, as in this case the necessity of much of this final discussion might have been eliminated.

The writer regrets that he has to take issue with Mr. Pruyn as to his (the writer's) experience, but it is only fair, in its relation to the paper, to state that it has not been confined to Brooklyn, but covers a somewhat large and varied experience in Manhattan, in dry, saturated, and subaqueous materials; and, further, covers a reasonably intimate knowledge of the action of earth pressures in excavations under railroad, municipal, and federal control, in the marshes and uplands of Delaware, in the red clays of Piedmont, Va., in the mountainous districts of Tennessee and Virginia, in the more or less saturated sands of Southern Georgia and Florida, and in the alkali



SHEETING, UNDER LIGHT PRESSURE, AT THE BOTTOM OF A DEEP TUNNEL IN MOIST SAND

plains and mountain slopes of Nevada and Utah; and he has invariably found that the same kind of ground acts in the same manner, no matter what its locality. Mr. Meem

Referring to Mr. Llewellyn's statement that the depth enters as a small factor into the calculations of the miners of Western Pennsylvania, it seems reasonable that this should be the case, particularly as in widening out drifts it is very essential that the depth be known. The experiments to be made by the Carnegie Steel Company (represented by Mr. Llewellyn) on interlocking sheet-piling should be watched with the greatest interest by engineers, particularly any experiments which relate to the bracing of trenches sheeted to great depths. That company's interlocking steel sheet-piling, with which the writer is familiar, is not especially well suited for experiments in proof or denial of the assumptions made in this paper, as will be shown later, as the longitudinal stiffness of this piling destroys somewhat the independent value of the pressures on each individual tier of braces. In other respects, the results of these experiments should be a most valuable asset to the knowledge of the action of material against piling of these types.

Mr. Moran's discussion is of interest. The writer has always used the straight-line formula noted in the paper for designing small retaining walls, and after arriving at the rectangle which balanced against overturning, he has cut in a diagonal face, thus giving a narrower top and wider base, which in itself established a factor of safety against overturning without increasing the weight of the wall.

Ordinarily, a wall designed on this rational basis will be found adequately strong against shearing, sliding, etc., but the proper calculation against failure along those lines may be readily made. Mr. White is not entirely correct in saying that the writer admits that his formula does not apply to retaining walls. What the writer wished to state was that a retaining wall designed by either or any formula would have practically the same general form, that is, it necessarily could not have increased thickness at the top. There would be, however, a difference in thickness and in the detail of the profile, according to the theory by which it was designed.

It should not be assumed that because a retaining wall stands, or because it fails, the theory of its design is right or wrong, just as the stability or failure of certain types of bracing does not prove or deny the correctness of any theory herein noted. For instance, the writer once, in his earlier experience, constructed a retaining wall of dry rubble masonry, 6 ft. high and 1 ft. thick, against a bank of sandy loam. By battering the wall toward the bank and tying in a header occasionally, he succeeded in getting a wall which, to the best of his knowledge, is still standing. He once observed in Washington a narrow trench, which had been excavated parallel with and alongside

Mr. Meem. of a car track to a depth of more than 25 ft., and which was standing absolutely with unsheeted vertical faces. He believes that, for some classes of earth, the protection of the face against erosion by light masonry or sheeting, is all that is necessary to hold it in place.

On the other hand, he can conceive a condition in which clay might slide on a moist well-defined seam probably at an angle of 45°, the pressure from which no wall of the ordinary type would resist; and, further, he does not believe that a wall retaining perfectly dry, sharp sand will have in reserve any factor of safety if designed by ordinary methods.

The writer has seen numerous instances, also, of cracked rangers near the top of a deep trench in which the bracing was tight. In each instance the bottom excavation was continued with entire safety after the weak members had been strengthened. These instances are given to show how essential it is to know one's ground, and, knowing it, to profit by the knowledge.

In its broad sense, it seems hardly necessary to adduce proof that, if sheeting is put in carefully and the bracing is made tight and strong as the work advances, the upper tiers of bracing will hold the upper strata of earth in a compact grip while the lower strata are being excavated and sheeted, and the writer would strongly advise anyone who has sheeted a trench close to the foundations of buildings and finds that the ground shows any signs of cracking, that his first care, after seeing that the sheeting is tight, will be to wedge up the braces which are directly in front of the foundations and nearest under the horizontal plane through their bottoms.

In the discussion, the difference between the writer and Messrs. Stern and others, seems to be principally the fact that these gentlemen assume that the accepted theories are infallible, and that practical conditions must be shown to accord with them, while the writer assumes that natural conditions are fixed, and endeavors to fit theory and the resulting formulas to them. A suggested careful reading of the paper, pages 10 to 13, inclusive, is the best answer which the writer can make to much of their criticism, and this he also believes will give a broader conception of the purpose of the paper than endeavoring to answer the quotations which Mr. Goodrich has separated from their environment. It is made clear in this portion of the paper that the writer does consider hydrostatic pressure, and also why it is that a flatter angle of repose (in materials "free from excess of water") gives less pressure on tunnels than material which takes a greater angle of repose. Mr. Stern takes the unique position of proving that the author is wrong by showing that he may be right by Coulomb's formula.

Many of those who discuss the paper seem to lose sight of the fact that sheeting is primarily intended to hold the material in place at all times in a compact grip, and is not simply a barricade to prevent

the occasional slumping of loose material. As to tight, strong, sheeting and bracing, these are not as difficult to get as many would seem to infer; and it should be a cause of reproach to the profession that poor bracing is so often seen. There should be no more valid reason for the failure of a braced trench or tunnel than for the failure of a concrete structure or bridge. Mr. Meem.

The discussions by Messrs. Cranford and Ketchum are of interest in showing different viewpoints of men who speak from wide experience.

Now, just a word to those who say that the assumptions made in this paper are contrary to all accepted theories. It appears to the writer to be beyond the range of successful contradiction, if the accepted theories are absolutely true, that there could have been no deep tunnels driven in soft ground by ordinary methods, and also that the sheeting of the bottom of deep trenches in sand would have been impracticable, as the sand would have burst through at the bottom before the sheeting could have been set in place; and further, that all theory is the result of practical experience, experiment and research, tabulated and formulated. For instance, even to-day, with their knowledge of materials, engineers rarely build that they do not destroy a portion in order to build the remainder more safely; and the writer contends that there have been no experiments of real value in connection with earth pressures, that is, as they relate to the assumptions of this paper. To have the experiments effective, they should be made with horizontal sheeting held in place by short vertical rangers, which, in turn, should be engaged by longitudinal rangers and braces, each tier of vertical braces being independent of the other, as shown in Fig. 72. The pressures on the braces could then be registered independently and accurately. The experiments should be made with absolutely dry, clean sand having a definite angle of repose, and with widely varying depths of trench. To summarize, for those who cannot find time to read the entire paper, the writer's conclusions, based on the classification made at the beginning of his discussion, are:

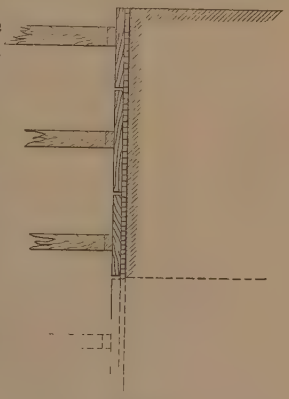


FIG. 72.

(1)—Class A materials arch themselves.

(2)—Class B materials:

(*a*).—Over tunnels these materials arch themselves, if there is sufficient depth overhead to give the required thickness

Mr. Meem.

of earth key; and they require bracing only to support the material below the intrados of this earth arch. The depth of tunnel is practically of no consideration beyond the limit noted above.

(*b*).—Around small shafts these materials arch themselves horizontally, leaving the bracing to carry only an indeterminate amount of loose material, as set forth in the paper. The depth of shaft is of no consideration if the shaft is small enough to admit of horizontal arching.

(*c*).—These materials arch themselves against the vertical or inclined face of a wall or braced sheeting, which face bisects the key and takes the place of half the arch. One spandrel wall of this arch is somewhere between the key and the slope line, depending on the nature of the material. The effect of this is to cause the pressure prism to assume the form of a triangle with its short leg at the top. The pressure against the sheeting is transmitted to the braces in the usual way, or it may be transferred to them by a series of vertical groined arches, leaving the sheeting and rangers to carry a uniform load of loose material "below" these groins.

Next to the angle of repose of the material itself, depth is the most important factor entering into the consideration of pressures of this class; and a depth of trench can be reached where it will be impossible to brace the top against the pressure which will develop in that area.

(*d*).—When the materials of Class B are submerged, account should be taken of the hydrostatic pressure, as set forth in the paper.

(3)—(*a*).—The pressures covered by materials of Class C should be calculated for the full hydrostatic head, based on the specific gravity of the aqueous material, and not its weight per foot together with its suspended matter.

(*b*).—It should be distinctly understood, however, that the presence of water in sand, by giving it cohesive qualities, adds to, rather than detracts from, the arching property of the material, unless the material is sufficiently soft or fine to be held in solution or suspension (as in pure quicksand).

(4)—The writer believes that the formulas given in the paper, whether new or standard, are absolutely safe for their respective conditions, but that a factor of economy may be introduced, in many instances, according to the experience and judgment of the engineer.

In conclusion, it is well, as Mr. O'Rourke suggests, to "stick to the big timber," when in doubt, but it is the function of the engineer to find where these big timbers properly belong and to eliminate them elsewhere, and it is the earnest wish of the writer, particularly in these days of forest conservation, that the Society shall not let this matter rest until the responsibility on these big timbers is properly placed and is defined so clearly that "those who run may read." Mr. Meem.

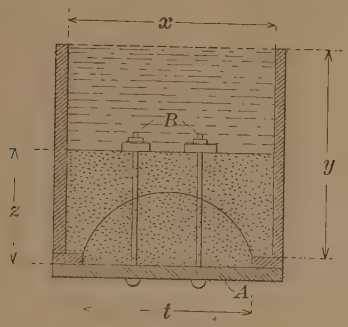


FIG. 73.

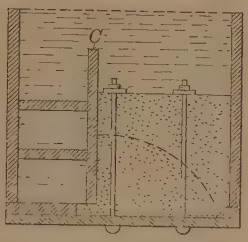
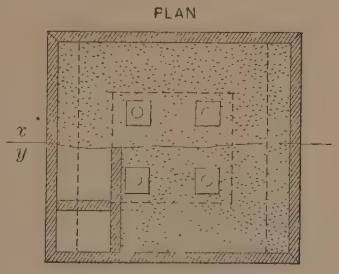


FIG. 74.

The writer offers the following description of some minor experiments which he has made since the paper and discussion were written. An 8½ by 8½ by 8½-in. box (inside measurements) was made, as shown in Fig. 73, the bottom being cut away, leaving only ¾-in. projections on two opposite sides and sheer faces on the others, thus making a hole in the bottom about 8½ by 7 in. A false bottom, 2 in. thick and 9 in. square, was then made, covering a little more than the outside area of the box, and four 6 by ¼-in. bolts were run through this, and engaged with 1-in. square washers and nuts as shown. It was found that absolutely dry sand would arch itself sufficiently to carry the false bottom and its own bottom load and a superimposed load, at a depth of 4½ in. (*z*, Fig. 73). Dry wheat with well-rounded grains was found to require a somewhat wider pier or haunch than ¾ in., therefore the width of the bottom opening was narrowed to 7 by 6½ in., when it was found that the arching effect of the wheat would carry its own bottom load at a depth of 5½ in. (*z*). A board was then inserted to represent sheeting, as shown in Fig. 74, and the sand was found to carry itself at a depth of 4½ in., showing clearly that arching took



x=Top View of Fig. 73
y= " " " " Fig. 74

FIG. 75.

Mr. Meem. place against the sheeting. Water was next introduced in sufficient quantity to allow it to seep through the crevices in the bottom, showing that the sand was completely saturated. It was found that at a depth of 5 in. the arching properties of the sand were such that it would carry its own and the added bottom load together with that caused by the box being filled to the top with water. It is believed that these experiments prove conclusively the principles set forth in the paper, although the scale of the experiment is somewhat too small to base upon their results any theoretical deductions other than the principles set forth in the writer's final summary.

A further pursuit of these experiments on a larger and more scientific scale is commended to those who may have the time and means to carry them out. The writer does not hesitate to say that, even on a scale large enough to be of practical value, he believes the results will be found to correspond with those obtained in the small box.

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INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1063.

Discussion.

RECENT PRACTICE IN HYDRAULIC-FILL DAM CONSTRUCTION.*

By Messrs. J. M. Howells, and J. D. Schuyler.

J. M. Howells, M. Am. Soc. C. E. (by letter).—Mr. Schuyler may be called the godfather of this system of constructing dams; he gave it the name, "Hydraulic-fill," more than ten years ago, and has since given it a greater name by the study and application of its principles in his wide practice, and also by the presentation of the results of his work to the profession. Mr. Howells.

As the establishment of a plant for hydraulic-fill dam building—either in the form of long ditches and flumes, or a pumping plant—is a very large item in the cost of the construction work which it serves; and as thorough foundation work is just as necessary, and often almost as expensive for a small dam as for a large one; and also, as water storage usually increases rapidly with the additional height of the dam; the assertion made by the author that, under proper conditions, such a dam can be safely built to a height of from 250 to 300 ft., has striking significance. The writer sees no reason to dispute this assertion, and, when such dams have been built, a great advance will have been made in their economy and usefulness.

The Location of a Hydraulic-Fill Dam's Drainage, and the Support of its Puddle Core.—The author describes two mishaps which for the time seemed almost to endanger two of the dams under discussion, *viz.*, the Crane Valley and the Snake River Dams. These mis-

* This discussion of the paper entitled "Recent Practice in Hydraulic-Fill Dam Construction," by James D. Schuyler, M. Am. Soc. C. E., was received too late to be printed with that paper in *Transactions*, Vol. LVIII, p. 196.

Mr. Howells. haps teach a lesson which should not be forgotten—namely, never to place the drainage system (intended to collect the seepage of the dam) in close proximity to the puddle core.

In the case of the Crane Valley Dam, the drain was a porous cement pipe moulded in place, without joints, founded on bed-rock, and covered to a depth of 2 ft. with a mixture of coarse sand and particles of broken stone of varying sizes. This was intended as a safeguard in case of any possible break in the porous drain, but when the latter was fractured by the emergency blast, as described, this sand and crushed rock, either on account of its deficiency or for some other reason, did not heal the break, but permitted the immature puddle core to leak away until stopped by other causes.

In the case of the Snake River Dam, the down-stream half of the entire structure is a drain in the form of a substantial rock-fill. This is separated from the hydraulic-filled puddle core by a double layer of wooden sheeting, but an imperfection in this sheeting caused a considerable quantity of the puddle core, while still immature, to leak away.

While it is known that the dam's weight ultimately squeezes out of the entire structure the water used for transportation, and permits the puddle core to mature by passing from practically a liquid to a solid state, yet, for security during construction, and as a greater precaution after maturity, the writer is of the opinion, as stated above, that no drain should be in close proximity to the puddle core. Neither should any perishable material form a partition between the two; for after the drain has delivered air to the wood and caused its decay (and especially if the wooden partition is nearly vertical and decays), will not the interstices in the rock-fill borrow from the puddle core and cause settlement? In the case in question, the nearly vertical wall of rock-fill which supported the plank sheeting was carefully hand-laid, and thus doubtless reduced to a minimum the size of the interstices adjoining the sheeting, and, with the rock supplied on the puddle side also, the chances of borrow were correspondingly diminished.

In most instances it would seem to the writer more desirable to place the wooden sheeting entirely within, and completely enveloped by, the puddle core, in which position, being entirely cut off from a supply of air, and being furnished with ample moisture, it would always serve its purpose as an extra precaution against leakage. The plank sheeting, if not carried too high, and if placed nearly horizontal or parallel to the surface upon which it rests, would require the seepage water to cross many joints, which, on account of the great pressure existing within the lower puddle section, would force these joints full of clay puddle and thus completely break the continuity of that seepage plane.

Mr. Howells.

Where the scarcity of material for hydraulic-fill requires the substitution of rock-fill, with the attending necessity for preventing the loss of immature puddle core through the rock-fill during construction, the writer would prefer a layer of coarse sand and gravel and broken stone of varying sizes, 5 or 10 ft. thick, depending upon local conditions. This layer should be as nearly vertical as the up-stream face of the rock-fill, and should be placed in position as the hydraulic-fill rises against it, thus supplying artificially more nearly a proper gradation of imperishable material for the complete support of the unstable puddle core. The expression, "more nearly," is used advisedly by the writer, as he believes that ideal conditions are secured only by hydraulic-fill throughout the entire dam, if proper materials are available; in which case imperishable materials, from the size of rock and boulders 1 ft. in diameter down to particles of clay of the fineness of flour, are systematically arranged so that perfect gradation and arrangement of the materials are effected by the agency of the flow of water under varying velocities and volumes. In which case the water-tight but unstable puddle core is supported in the center by what may be called two gritty dams reclining against and upon it, the latter so composed by gradation from fine, next to the puddle core, to coarse on the outer faces, as to afford a perfect sequence of support; and yet, even among the coarse gravel and rock composing the outer third of the dam, the interstices are filled so solidly with smaller stones, gravel and sand, as to afford not only the high friction needed for stability, but are locked together so compactly as to be entirely vermin proof.

As to the exact position, then, which a drain should occupy, the writer can suggest no fixed rule, but believes that, in the majority of cases, the dam site in the stream-bed portion will be found covered with a bed of sand, gravel and boulders of greater or less thickness. To excavate and carry away this bed of gritty, porous material from under the down-stream half of the dam would be an expensive error, when it is so greatly needed in its natural position as a drain, or collector of seepage for delivery to a drain, to be bedded as deeply within its mass as is convenient for a gravity outlet below. The drain pipe in such a case should run longitudinally with the dam and far enough from the center to be under the lower porous third of the dam; and, when the hydraulic-fill begins, care should be taken not to cover the down-stream third of the dam site with water-tight material and thus cut off the downward approach of seepage toward the drain.

For the portion of the base of the dam which lies without the stream bed, these natural conditions should be imitated artificially, and this the writer also recommends for the stream-bed section of other dam sites where no beds of gravel and boulders exist.

A Suggested New Departure in the Delivery of Material.—In most instances in the writer's experience, the completed work would have

Mr. Howells.

been more economical if convenient material had not been so scarce; and the possible material for gravity delivery into the dam has often been found indurated and difficult or expensive to put into liquid suspension; also, the grade lines are almost invariably too flat for the delivery of the rock so essential to ideal construction. On the other hand, deep deposits of ideal material—scarcely ever indurated—are almost invariably found in the proposed lake bottom or on the lower slopes about the dam site. Furthermore, a plant for reaching lower material is far cheaper to install than one delivering water under high pressure to a high elevation. All this has led the writer to determine to make an innovation in the next work of this kind where these ordinary conditions prevail, that is, to deliver all material by a heavy gradient to the lowest possible point in the center of the dam site, and there to construct a masonry well or tower, of generous size, extending from the bed-rock up to the top of the dam.

The progress of this tower as it rises would precede the hydraulic-fill sufficiently to yield a heavy gradient from its top in all directions to all parts of the dam; and in this tower would operate an extension elevator dipping both water and hydraulic-fill from a smooth metal-lined sump below, into which sump the hydraulic-fill would be supplied by a culvert connecting with the regular heavy-grade sluiceway.

If no use could be found for this tower as a gate chamber in the completed dam, the cheaper method of a temporary timber tower reclining upon the up-stream face of the dam might be advisable.

An Instance where Hydraulic-Fill was Indispensable.—The writer first applied hydraulic-fill to dam building about fifteen years ago, in connection with an earth dam, 87 ft. in height, built in the cañon about 3 miles east of Santa Fé, N. Mex.

The material for the dam was being delivered and spread by wheel-scrapers, additionally spread by dragging a railroad iron, sprinkled with water wagons, and then puddled by the constant travel of a flock of 150 goats. One end of the dam joined a continuous exposure of limestone ledge while the opposite end was founded upon a drift deposit, into which test-pits had been sunk to bed-rock and through which a continuous puddle trench was cut.

As the dam rose against its abutments, the puddle trench was excavated in advance, and, on the bed-rock end, the ledge was also excavated into more solid material. When the structure had attained half its proposed height, the trench in the limestone ledge suddenly developed caverns, often of the size of badger holes, which, upon being followed, led off into the mountain. A re-examination of the adjoining mountain against which the reservoir water was to stand revealed good solid rock, and, on the flatter slopes, stable beds of drift deposit. There was a limit to which this cavernous limestone could be followed into the mountain, and, having already a deep excavation,

the writer determined to apply the hydraulic method for filling the caverns and sealing the dam's connection with that abutment. Mr. Howells.

All water wagons were put into service, and most of the mules and all the goats were given a vacation which, fortunately for the economy of the work (and perhaps unfortunately for the mules and goats), was of short duration.

Water was hauled and suddenly released on the concave top of the dam, which was rather hopper-shaped next to the cavernous limestone trench, as that part of the fill had been left lower than the remainder of the work. It rushed into the caverns, carrying with it a load of sandy clay supplied from a loose pile at the entrance of the trench. At first those of the caverns which had soft walls were even enlarged, but the hydraulic-fill was continued, and finally ceased to be swallowed up, but would still drain away rapidly; at no time, however, did it issue from the sides of the mountain below. The fill was finally complete, and even clear water would no longer sink, but stood in a pond on that end of the dam ready to be absorbed by the new material delivered by the wheel-scrapers. From this level up to the top of the completed dam, the hydraulic test was applied, and once again at a higher level similar caverns developed and were filled.

This dam has now been in use fourteen years, and has given no evidence of leakage, and the writer, believing that no other economical method could have overcome the difficulty of its connection with this abutment, then became convinced that the time had arrived for building a hydraulic-fill dam complete, which, on account of the great responsibility involved, he had theretofore hesitated to inaugurate. Accordingly, the method was put into practice in the dams constructed immediately after the Santa Fé Dam, as described by the author.

The writer, as firm a believer and advocate of the hydraulic-fill dam as ever before, believes he sees a still widening field for its usefulness, and is about to begin in Japan the construction of the highest dam yet undertaken.

J. D. SCHUYLER, M. AM. SOC. C. E. (by letter).—The belated discussion of the paper on hydraulic-fill dam construction* by Mr. Howells is a welcome contribution, and calls for a few remarks in reply. Mr. Schuyler.

It is interesting to note that Mr. Howells is in agreement with Mr. F. P. Stearns in preferring the homogeneous type of purely hydraulic fill, in which the exterior layers of rock, gravel, and gritty, friction-bearing materials rest upon and are supported by the interior mass of clay puddle, which has been separated from the material, found heterogeneously mixed in the borrow-pit, by the varying velocity of the water. The combination type of rock-fill, backed up by a hydraulic-fill upon the up-stream side, used so successfully in the dams on Snake River, in Idaho, has much to recommend it, however, and is ex-

* *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 196.

Mr. Schuyler. tremely useful in certain localities where rock is plentiful and earth is comparatively scarce.

In the case of the Zuni Dam, in New Mexico, just completed, experience has shown that this type of dam was the only one which could have been built, under the peculiar conditions attending the construction. The rock-fill was built first, as a hand-laid dry masonry wall, with a down-stream slope of $1\frac{1}{4}:1$ and an up-stream slope of $\frac{3}{4}:1$, leaving a gap of 60 ft. or more in the center. During construction, an unprecedented flood, exceeding anything which had occurred for 200 years, passed down through the gap without materially disturbing the work, although the erosion deepened the channel throughout about 10 ft. Again, after the gap in the rock-wall had been partly filled and hydraulic sluicing had begun, succeeding floods occurred in which the water rose to a considerable height back of the wall and found its way through, without doing more harm than to cause a settlement of about 18 in. for a distance of 60 or 80 ft. in the center. In completing the fill, the distortion thus caused was removed to some extent, so that it is now scarcely noticeable to the eye.

Had the original plans for building the dam as a homogeneous hydraulic-fill not been changed to the composite type, by the advice of the writer, the work would have been totally destroyed by the floods, and doubtless would have led to the abandonment of the attempt to build a dam under such discouraging conditions. The completed structure, however, is one of the most substantial and satisfactory dams in the West. The hydraulic filling was placed, as described in the paper, against the face of the rock-fill, in the manner advocated by Mr. Howells in his discussion, as preferable to a wooden diaphragm in the rock-fill such as was used in the Snake River Dams. In order to prevent the soft mass from flowing into the rock-fill and escaping down stream, it was necessary to build a dry embankment (as narrow as possible, and just wide enough for a team to drive over) against the face of the rock embankment. This made the work proceed somewhat more slowly than if there had been only one levee to build, and it was possibly more expensive than with a wood core as a stop for the immature clay puddle.

It may be of interest here to note that the total time of sluicing at the Zuni Dam was about 800 hours, during which time 40 000 cu. yd. were moved with a stream of 2.5 cu. ft. per sec., giving an average load of 15% of solids. The cost of the work, including fuel and labor, building levees, etc., averaged 12 cents per cu. yd. The best day's work was 750 cu. yd. deposited in 8 hours.

The writer can see no objection to the wood diaphragm in the center of the rock-fill, with a puddle of clay filling the voids in the rock on one side, even assuming that the wood may finally decay. It will surely last long enough to permit the clay to solidify and become

mature as a puddle core sufficiently solid to stop leakage effectually. As the wood decays, it will be constantly under pressure from both sides, and any softening or yielding of the wood must be taken up by the gradual settlement of the mass around it. The function of the wood will have ceased as soon as the clay puddle becomes mature, and, in the writer's opinion, this must occur before decay takes place. On general principles, however, it would be desirable, as Mr. Howells points out, to have the wood rendered imperishable by being embedded in the clay puddle. This plan is being adopted in the building of the Nuuanu Dam, now under construction for Honolulu in accordance with the writer's advice. This is a hydraulic-fill dam, 1 700 ft. long, and 75 ft. high in the center, with a section of rock-fill on the down-stream side, covering only the stream channel, about 150 ft. long. This rock-fill is built up as a dry wall, vertical on the up-stream side, which is placed 10 ft. below the line of wood-diaphragm that runs through the entire length of the dam, the space between being filled with selected clay puddle, rammed in moist. The purpose of the rock-fill in this case was to give more effective drainage to the lower toe of the dam. This question of drainage is an extremely important one, and the writer fully agrees with Mr. Howells that any artificial drainage provided should not reach in as far as the center core of the dam, but should be confined to the outer third, leaving a stable mass outside, and thus avoiding the danger, encountered in the Crane Valley Dam, of losing a part of the unsettled, immature core material by flowing through the drains. Mr. Schuyler.

In building the Silver Lake Dam, in Los Angeles, Cal., referred to in the previous discussion, temporary drains have been provided by driving six 2-in. pipes, some 60 ft. horizontally into the down-stream face of the dam. Each pipe is provided with a Cook well-point and strainer at the end. They are about evenly spaced over a length of 200 ft. of the higher part of the dam and from 6 to 10 ft. above the surface. The greatest quantity of seepage gathered from any one of these pipes amounted to 2 gal. per min. This has gradually diminished to one-fourth this quantity. One or two pipes have ceased flowing, and the others have a very small flow. This dam is finished to a height of 50 ft. (Elevation 444), within 6 ft. of the top, at the present writing (November 5th, 1907). From Elevation 436 to Elevation 442, a careful account of cost was kept, on a delivery of about 8 000 cu. yd. The average cost of all labor, materials, fuel, pumping, etc., was 16 cents per cu. yd., including the cost of the levees. Three pumps were used; one for supplying the jets for loosening the earth, under a nozzle pressure of from 70 to 100 lb. per sq. in.; the second for pumping the liquid earth and water through 3 500 ft. of 8-in. pipe, and a third, or "booster," pump, located midway on the discharge pipe, to assist in relieving part of the load on the main pump, the latter having a press-

Mr. Schuyler.

ure of about 40 lb. and the "booster" about 10 to 12 lb. per sq. in. About 10 lb. of this head represented the lift to the dam; the remainder was the friction in the pipes. At present, the discharge pipe has extended to a distance of 4 000 ft. from the dam. All material for the dam (146 000 cu. yd.) has been taken from below the water line of the reservoir. During the period noted, when the cost account was kept, the delivery of material averaged 500 cu. yd. in 9 hours, and, as the volume of water pumped was about 2.5 cu. ft. per sec., the solids averaged nearly 17 per cent.

Recent reports from the Santo Amaro Dam, in Brazil, show an average delivery of 5 700 cu. yd. in 24 hours, at a cost for labor and materials of 8.5 cents per cu. yd., exclusive of power, which would add about 2.1 cents per cu. yd. if the value of the electric current furnished were taken at 0.5 cent per h. p.-hour.

The highest rate of delivery of mixed rock and clay yet reported at the Necaxa Dam, with one hydraulic monitor, discharging about 17 cu. ft. per sec. under a pressure of 130 lb. at the nozzle, is about 8 000 cu. yd. in 24 hours. The highest percentage of solids delivered is about 20. The minimum cost for labor alone is 2.5 cents per cu. yd. Rocks of all sizes, up to 1 ton in weight, are delivered through the flumes to the dam, at a velocity of 20 ft. per sec. on an 8% grade.

A dam has recently been built near Anaheim, California, for the Anaheim Union Water Company, by Mr. H. Clay Kellogg, which has many interesting features in its construction, as about 80% of the volume has been placed by the agency of water. The dam is 47 ft. high, 300 ft. long on the base, 800 ft. long on top, with an up-stream slope of 3.5 to 1, a down-stream slope of 2 to 1, a crest width of 16 ft., and containing about 100 000 cu. yd. The ground-sluicing method was used for the lower half of the height, the material being loosened by plowing and by picks and bars. The water used was from 6 to 8 cu. ft. per sec., and was divided into two sluicing streams, the material being conveyed in flumes laid on grades of from 4 to 7 per cent. The average cost of ground-sluicing was about 8 cents per cu. yd. When the dam reached a height above which it was no longer practicable to secure the necessary gradients for conveying the material by gravity, a single-stage centrifugal pump, operated by a gasoline engine of 60 h. p., was installed to deliver the sluiced material through an 8-in. pipe, 800 ft. long, to and along the dam. This pump has a capacity of 3 cu. ft. per sec. It was located at one end, and 20 ft. below the crest of the dam. A second centrifugal pump, with a capacity of 0.5 cu. ft. per sec., was installed for supplying the hydraulic stream for cutting the bank, under a pressure of 35 lb. per sq. in., delivered through a 1-in. nozzle at the end of a 2-in. hose. This sufficed to do the mining, and clear water from a ditch was added, to the extent of 2.5 cu. ft. per sec., to carry the loosened earth to the lower pump. In this manner about

600 cu. yd. were delivered daily in 10 hours at a total cost of about 12 cents per cu. yd. The material consisted of clay, sand, and gravel. The side levees were built up by scraper teams. The water used in the work was drained back into the reservoir by a stand-pipe connecting with the outlet culvert. No core-wall was used in the dam. The reservoir is intended for irrigation service. Its capacity is 51 000 000 cu. ft. The work was begun in February, 1907, and was recently completed. Mr. Schuyler.

A gratifying indication of the rapid spread of the hydraulic-fill process of dam-building is afforded by the fact that Eastern engineers are beginning to adopt it. The writer has learned of a very large dam being planned for the Cambria Steel Works at Johnstown, Pennsylvania, where a De Laval turbine engine is to be direct-connected to a three-stage centrifugal pump for discharging 10 cu. ft. per sec. under a head of 150 lb. per sq. in. at the nozzle for hydraulic sluicing. The dam will be 12 miles from Johnstown, is to be more than 100 ft. high, and will contain 350 000 cu. yd. Including deep and expensive foundations, large culverts for handling excessive flood discharge, and other accessories, it is estimated to cost $500 000.

The process is being considered for the construction of two dams for Springfield, Massachusetts, and plans are also being matured for the use of this method for the construction of the Standley Dam, near Denver, by J. G. White and Company. This dam will exceed all previous records for volume of material to be moved, as it is to be 140 ft. high, 8 900 ft. long on the crest, and contain nearly 7 000 000 cu. yd.

For boldness of plan and dimensions, however, the hydraulic-fill dam projected by the Buffalo Basin Irrigation Company, of Wyoming, takes the palm. This dam is to create a reservoir for the irrigation of 150 000 acres of land, and is said to be planned to a height of 250 ft. and to contain nearly 15 000 000 cu. yd.

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TRANSACTIONS.

Paper No. 1064.
Discussion.

THE NAVAL FLOATING DOCK—ITS ADVANTAGES, DESIGN, AND CONSTRUCTION.*

By Messrs. L. F. Bellinger, H. T. Hansson, and Leonard M. Cox.

Mr. Bellinger.

L. F. Bellinger, M. Am. Soc. C. E. (by letter).—The author takes up both sides of the question of floating and graving docks, and leaves so little room for controversy that it is very difficult to discuss the paper. One point which is not brought out relates to locomotive cranes. On graving docks the usual capacity of such cranes is from 40 to 45 tons; and they can travel all around the ship, except across the caisson. With a floating dock, however, very heavy work, such as propellers, shafts, etc., can be brought on a floating derrick having very much greater capacity than locomotive cranes. This floating derrick, having a capacity up to 100 tons, can come alongside the end of the floating dock and land the necessary piece of machinery close to the bow or stern of the ship, from which it can be rolled along the deck and hoisted into position by rigging and tackle from the bow or stern of the ship. With the locomotive crane, in many instances, the floating derrick has to bring the piece of machinery to the navy yard and land it on the wharf; from this point it is taken by the locomotive crane to the ship.

In the Algiers dock several matters needing improvement were attended to soon after the dock went into commission. One of the difficulties was to get rid of the water on the decks of the pontoons. It was recommended that new floating docks should not have depressed floor plates, but that the deck plating should be flush; it was also

* This discussion of the paper by Leonard M. Cox, M. Am. Soc. C. E., entitled "The Naval Floating Dock—Its Advantages, Design, and Construction." was received too late to be printed with that paper in *Transactions*, Vol. LVIII, p. 97.

recommended that there should be a camber in the deck of the dock so as to carry the water to one side. It was found later that the water could easily be controlled by cutting **V**-shaped openings in the keel-block bearers, thus giving the dock a list of from 3 to 5 in. to one side. This permitted all the water to run into a little gutter on one side of the dock next to the altar. Then, with a fore-and-aft slope of from 5 to 12 in., the water ran from the deck readily. With a lateral list on the *Dewey,* the water can be taken care of without any camber. This permits the dock to be constructed much more quickly. Another objection to the way in which both floating docks are constructed is the location of the manholes. In each case, the bilge-block slides cover and obstruct the manhole covers. This makes it necessary to take up the slides in order to examine the interior compartments of the pontoons or to work in them in painting, clearing out mud, etc. Mr. Bellinger.

The author is in error in his statement regarding the operating expenses of a dry dock, as compared with a floating dock. Instead of only one man being required in a dry dock pumping plant, there are, in the pump pit between Dry Docks Nos. 2 and 3 at the New York Navy Yard, three men during the day watch and two during the night watches; one is required at the switch-board, and the other to handle the valves.

With regard to the flexibility of the dock allowing it to assume the shape of the keel, it is believed that this will be of no particular value, because, if there is any unevenness of the keel, or any uneven distribution of the weight, either in the floating dock or the graving dock, the keel blocks will be crushed. In some cases, the keel has forced itself into the blocks 4 or 5 in.

The question of traveling cranes or derricks was taken up at various times in connection with the Algiers dock. It was found exceedingly difficult to arrive at any satisfactory conclusion. In any crane of the gantry type traveling along the side-wall, the legs would have to be high enough to clear the smokestack in the center of the side-walls. In addition to this, all extra weight placed on a floating dock, whether in the form of a machine shop, air compressors, derricks, or cranes, requires additional work every time a ship is raised. This increases the operating expenses of the dock, and the additional weight has to be maintained, because the actual increase of work is equivalent to the weight of this additional material raised the same vertical distance as the dock. A very important change was made in the Algiers dock in substituting steam capstans for the steam winches, and adding two extra ones; thus one winch at the bow now handles the anchor chains, and three capstans handle docking lines on each side-wall. One additional improvement, not yet adopted, is a change in the bridges leading from the shore cylinders to the dock, making the profile of the top

Mr. Bellinger. surface a straight line, and arranging the bridges in such a way that small cars running on standard-gauge railroad track could run down the wharf and over them directly to the gangway openings.

A very important point, taken up by the author in a general way, is the cost of painting. As it is thought that some detailed figures on this subject will be of interest, they are presented here.

The actual cost of one complete self-docking and painting was:

Labor	$11 529.32
Material	6 443.37
Total	$17 972.69

Such painting should be done every two years if the dock is in fairly constant use.

The painting of the upper decks of pontoons and side-wall, which should be done every year, costs:

Labor	$577.63
Material	860.58
Total	$1 438.21

The total yearly cost of painting, etc., therefore, is $10 425, or $1\frac{2}{7}\%$ of the cost of the dock. For four years the cost of repairs on this dock has been kept. The average annual cost has been:

Body of dock	$663.00
Pumping plant	96.00
Blocks, chains, fenders, and other accessories	371.00
Painting, total annual cost	6 550.00
	$7 680.00

or less than 1 per cent. It is probable that another coat of tar-paint and a self-docking is now due.

A history of the tar-paint used on this floating dock is quite interesting. In 1891 there was patented a compound called "Pyro-paint," the formula for which was not very rigid. It contained very small quantities of salt, red lead, ochre, asbestos, etc., lime and cement; in all, $\frac{1}{2}$ lb. of these small ingredients to 1 gal. of coal-tar. Robbed of all mystery, this meant an alkali, a thinner, and coal-tar.

As the Morgan line of steamships (running to New Orleans) had been painted successfully with plain coal-tar for some time, experiments were begun, in October, 1902, by A. C. Cunningham, M. Am. Soc. C. E., Civil Engineer, U. S. N., at the New Orleans Naval Station, with various mixtures of coal-tar, Portland cement, lime, turpentine, and kerosene. Strictly speaking, this coal-tar, having been pro-

duced in the manufacture of water gas, should be called gas-tar. In these experiments it was found that lime could only be used as a putty. A report by Mr. Harold L. Hoyt, in January, 1903, states that kerosene is far superior to turpentine, both as a thinner and as a dryer. The kerosene mixture covered surfaces more completely, dried more quickly, and presented a harder and more glossy surface. As the result of those experiments, the best mixture was found to be 8 parts tar, 1 part Portland cement, and 1 part kerosene; all measured by volume. The chemical analysis showed no acidity in the mixture, therefore, nothing is to be feared in connection with applying this compound to steel. This was called the Yards and Docks tar mixture at the New Orleans Naval Station, but it is now called the Cunningham tar-paint. Mr. Bellinger.

In connection with the floating dock at New Orleans, the history of the painting is interesting. Before April, 1902, the iron oxide paint which had first been applied to the structure had disappeared to a very great extent, so that in April, 1902, the contractor painted the dock with red lead and white zinc. In June, 1903, in the interior of the dock, the red lead below the water-line was good, but where the surface was alternately wet and dry (caused by the operation of the dock in docking vessels) the paint was in poor condition; the oxide of iron paint on the side-walls required renewal, and what remained on the dock came off easily with a brush. In December, 1903, after two years of use, it was found that the bottom of the dock contained 5 in. of mud, and the sum of $250 was required to remove it. This was done by steam siphons, with water-jets to stir up the mud.

In the fall of 1903 the work of scraping, cleaning, and scaling the mass of structural steel inside the pontoons and side-walls was commenced. It was found that 25% of the cost was required for painting, and 75% for scraping. Considerable black mill scale was found on the steel plates. In order to get this off and remove the rust scale which adhered very tightly to the plate, a diluted solution of commercial sulphuric acid (between 30 and 40%) was used. The acid was applied to the plating with frayed rope ends. After being left on the steel for from 20 to 40 min., depending on the tenacity of the scale, the plating was washed down with a hose. This process softened the scale so that it was easily removed with ordinary paint scrapers. Painting followed this pickling operation.

The self-docking of this dock was estimated to require a force of twenty laborers, four machinists, and two shipwrights, at a total cost of $57 per day. Docking and painting inside and outside the center pontoon and side-walls below the water-line required two months. Docking and painting the end pontoons inside and outside required one month. The force was about 100 men. On October 1st, 1904, the cost of painting all except the lower 7 ft. of the pontoons and side-walls was $4 100 for labor and $2 700 for material. On this dock two

Mr. Bellinger. or three dozen kinds of paint were used in competition with each other, but in 6 months' time all had failed from a line about 5 ft. below the deck. Above this line they were in good condition. In trying to find a paint which would last, cement grout was applied to the steel with a brush, and it was found that 1 200 sq. ft. of surface could be covered with 1 bbl. of cement. It was feared, however, that the vibration of the dock while the engines were operating would cause the cement coating to crack off; it was also found that, under these conditions of alternate moisture and dryness, the tar-mixture lasted just as well as any paint applied. The relatively poor showing of all paints tested on this dock is given in the following extracts from an official report in October, 1904:

"The fact that the new paint is in bad condition, is, in my opinion, due to the manner in which the work was carried on. It was the desire of all officers present to accommodate shipping in the port as much as possible, in the use of the dry dock. In following out this policy, the work of scraping and painting the dry dock was subordinated to the necessities of the shipping of this port, and was applied whenever the dock was not in use. The effect of this was as follows: While the dock was in use the paint was thoroughly soaked in water. When the dock was raised for painting purposes, the water in the pontoons was pumped as low as possible which permitted the paint already applied to dry out thoroughly. The series of wettings and dryings mentioned above has caused the paint to lose its adhesion to the steel, leaving it in the condition first described.

"The reason that the red lead paint mixture applied by the contractor stood so well for about two years is believed to be due to the fact that paint was applied all over the dock at one time and has been kept wet practically ever since it was applied. This paint, though in good condition last fall, on account of its being alternately wet and dry, caused by the docking of ships during the painting of the dock, has lost all of its valuable properties, and the whole work, on the inside of the pontoons, needs immediate attention.

"The condition of the steel, owing to the thoroughness with which the scraping was carried on, is very good, so that, if taken up at once, there should be very little labor necessary, outside of the painting."

While the tar could be applied, cold, without trouble, by using a small additional quantity of kerosene, it was found easier to work by applying the tar hot and using less kerosene. The quantity of cement used was between 1 pint and 1 quart per gallon of tar, the best way of making the compound being to incorporate the cement with the kerosene, and then stir the mixture thoroughly into the tar when it is to be used.

The result of all these experiments was the adoption in January, 1905, of the following: one coat of paint consisting of 2 parts red lead and 1 part white zinc was applied to the steel work, then a wearing coat or surface coat of the Yards and Docks tar-mixture, as above described, was applied on the red lead, inside and outside of the

pontoons below the water-line. On the pontoon decks and side-wall decks one coat of red lead and white zinc was applied, with a wearing coat of red lead and graphite paint. In the engine-room two coats of white lead and white zinc were applied on the machinery, and for 6 ft. up on the engine-room walls; above that height one coat was applied. Mr. Bellinger.

In addition to its use in New Orleans, La., this Yards and Docks tar-mixture has been used by Civil Engineer A. C. Cunningham, U. S. N., on the old tin roofs on the buildings at the United States Naval Academy, Annapolis, Md. It was very successful in stopping leaks, even when the tin sheets were full of pin-holes. In the New York Navy Yard, tin roofs and galvanized-steel roofs have been painted with this tar-mixture for the past year with most excellent results. The caisson of one of the dry docks was painted with one coat of red lead next to the steel and a wearing coat of the tar-mixture over the red lead. This was exposed to the salt water of the East River for nine months and then examined. At the water-line, where wave action, ice action, etc., had its full effect—that is, between low water and high water—the tar-mixture was worn through to the red lead, but below the water-line it was intact, though the glossy black surface was considerably dulled, giving a sort of brownish appearance. On the tin roof of one building—a blacksmith's shop in the New York Navy Yard—there were some valleys, over some of the machinery in the building, which were very leaky. Some holes in this roof were as large as a silver 25-cent piece; these were plugged with a kind of putty, made with a large proportion of tar and cement and a very small proportion of kerosene, and then the valleys in the tin roof, which were full of smaller holes, were painted with the regular tar-mixture. This was done during the fall of 1906, and examined during April, 1907. It was feared that the tar would crack during the cold weather and permit leakage. These old valleys are in perfect condition to-day, as far as leakage is concerned, and not a single complaint was made during the entire winter. On the steel piles and steel cylinders of a coaling wharf at East Lamoine, Me., this tar-mixture was applied to the steel between high and low water, the tide coming up over the paint before it had thoroughly dried. The report states that ice and tide action have gradually worn off the paint, but that "it wears better than anything else that we have given a thorough test."

H. T. Hansson, Esq. (by letter).—One of the objections commonly urged against floating docks is the expense of maintaining the depth of water under their bottoms in situations where silting takes place. It is, however, an easy matter to fit a floating dock with silting pumps so that the dock itself can maintain and even increase the depth of water without removing it from its site or stopping its regular work. Such an arrangement is proposed for the 8 000-ton dock designed by Lyonel Clark, Esq., for Aberdeen. Mr. Hansson.

Mr. Hansson. As regards the handling of weights, it does not seem that the floating dock is worse off than the graving dock. It is said that cranes on top of the side walls would interfere with the handling of lines, but this is hardly the case, as any handling of lines takes place only when a ship is entering or leaving the dock, at which time there would be no occasion to use the cranes, and they could be run to the forward end of the dock, thus leaving the side wall decks free. In most modern shipyards, the tendency is to install powerful floating cranes for handling heavy weights, and to such cranes the floating dock is much more accessible than the graving dock.

In emergencies, many dockings have taken place in floating docks that never would have been undertaken in a graving dock of similar size. A case in point was when the floating dock at Barrow lifted the *Empress of China* to replace a propeller. The Barrow dock is 242 ft. long, over keel blocks, and the *Empress of China* is 456 ft. over all, thus leaving an overhang of 107 ft. at either end. The steamer *Shawmut,* of 9 606 tons, was docked on the floating dock of Moran Brothers, at Seattle, Wash., for the purpose of replacing the sternpost. This floating dock is of 2 700 tons capacity; consequently, it was out of the question to lift the whole ship, and it was decided to lift only the stern. In order to do this, a coffer-dam, conforming to one of the after frames of the ship, was built at one end of the dock. The stern of the steamer was then landed on the keel blocks and lifted, while the forward part of the ship was all outside the dock.

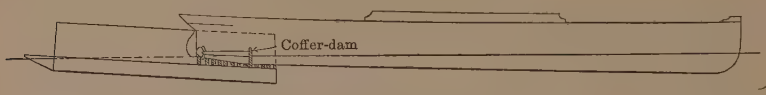


FIG. 1.

It is thus seen that, in extreme cases, a floating dock can perform work that would require a graving dock several times larger.

The advantage of one type over the other, thus far, has been mainly dependent upon the site, and it seems that conditions in the United States have been more favorable to floating docks, there now being 86 floating docks, as compared with 74 graving docks; while, for the future, the advantages seem to be all in favor of floating docks, on account of the greatly increased draft over keel blocks required by modern ships. An increase of draft, of from 30 to 40 ft., over blocks in a floating dock is not accompanied by any structural difficulties, and the increased cost is very moderate, while such an increase of depth of water over sills in a graving dock will increase the structural difficulties and the cost enormously.

As one of the advantages of the floating dock for military purposes is its mobility, the permanent equipment of the naval dock ought to be

such as to render the dock ready for sea-going without any extensive additions. In view of this, a short account of the additions made to the equipment of the *Dewey* to fit it for its voyage to the Philippines may be of interest. Mr. Hansson.

The *Dewey* was delivered to the Government anchored in the Patuxent River, where the dock was prepared for the tow. The moorings consisted of eight 4 000-lb. mushroom anchors, two at each corner, with 45 fathoms of 2-in. chain to each. There was, however, no provision made for working these anchors, consequently a new set of ground tackle was supplied, consisting of four 9 000-lb. stockless anchors with 225 fathoms of 2¼-in. chain for each. For the working of these, a windlass was installed on the main deck amidships, and chain lockers were built underneath. For housing the anchors, two 24-in. hawsepipes were provided at either end, bracketed outside of the pontoons. A similar arrangement was also made for housing two of the mushroom anchors at the after end of the dock. Four towing bridles, consisting of 45 fathoms of 2½-in. chain for each leg, were supplied. For the belaying of these, four pairs of 24-in. bollards were added, making, with those already on the dock, a total of sixteen, or two to each leg of the bridles. A wooden trestle and derrick were built at the after end of the dock, from which the bridles were to be handled when it was desired to tow the dock stern first. Wireless telegraph apparatus and Ardois light signals were also fitted. Besides this, a number of pad-eyes and links were riveted on the main deck for lashing purposes. Davits were provided for two 26-ft. whaleboats, new guys were put on the smokestack, extra quarters fitted, etc.

From the experience gained during the tow, the following suggestions may be made:

The original design for a naval floating dock ought to provide means for working the ground tackles and housing the anchors.

The capstans on the dock had only one speed, which proved a great handicap when the tow lines were sent over from towing ships to the dock after the dock had been adrift. In order to do this, the steamer that was going to tow next ahead of the dock would bear down on the dock from windward and pass close enough to throw a heaving line on board. Bent to this would be a 3-in. line, to which was bent a heavier line, usually 6-in., which finally was bent to the towing wire on the drum of the steamer's towing machine. As the steamer could not run as slowly as the dock was drifting, she had to pay out a whole lot of line in passing the dock and then stop to leeward for the dock to catch up with her. The capstans were not able to pull in the line as fast as the dock was drifting, in consequence of which the line would form a bight, the resistance of which in being towed through the water was such that the greatest care had to be taken not to part line, thus making the connecting up of the tow a very tedious proceeding. There-

Mr. Hansson. fore, it seems that some of the capstans ought to be provided with a high speed.

The wireless telegraph proved of great value during the tow, and, as a naval dock might be moved to a site where there would be no other means of communication, it would seem desirable to make such an outfit a permanent part of the dock's equipment.

Chafing gear ought to be provided for all sharp corners. The manila hawsers used in passing the towlines were generally so badly chafed that they were of no value after being used once, which caused the expedition to be short of small lines after the dock had been adrift a couple of times.

The wooden trestle built on the *Dewey* was not used for the purpose intended, as it was found better to handle the bridles from the main deck. At the same time, a swinging bridge aft, like the one provided forward, would doubtless prove of great value in that it would facilitate communication between side walls both at sea and with a ship in dock, as well as being useful in centering a ship.

It would, without doubt, greatly reduce the towing resistance if the ends were given some less resistant shape than square, and the writer thinks the scow shape would be preferable to the pointed ends sometimes used on docks. A floating dock will always make an angle with the course in any wind as long as it is not from dead ahead, so that, if a dock has pointed ends, the sides of these would always be more or less square against the direction in which the dock is moving, whereas, if the ends are scow-shaped, the angle would always remain the same, besides which, the scow shape has the advantage of not reducing the deck area, as would be the case with pointed ends.

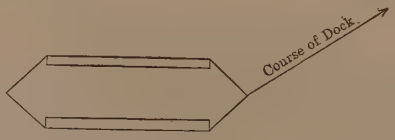


FIG. 2.

Besides, for towing purposes, it is advantageous to give the ends some kind of shape, because it greatly reduces the longitudinal bending moment, thus subjecting the dock to lesser strains both when docking ships and in a seaway.

Mr. Cox's explanation of the surging of the bow section of the *Dewey* while being submerged during the self-docking trials, that is, that it was caused by the unsymmetrical water line of the pontoon, does not seem to the writer to be correct. It is more probable that the surging was caused by the current which was running rather strong at the time and the effect of which was not anticipated by the valve man.

Mr. Hansson.

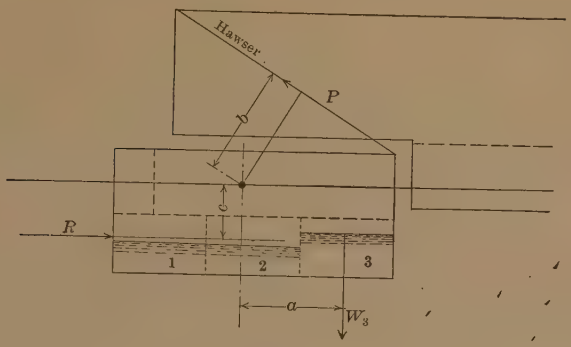


FIG. 3.

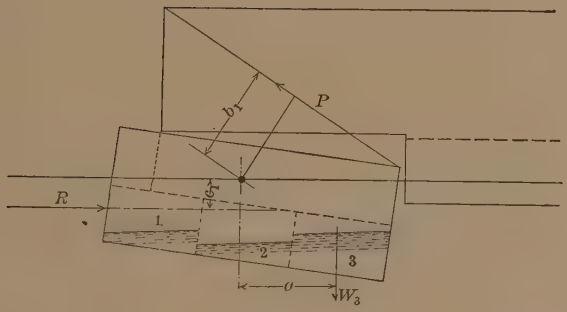


FIG. 4.

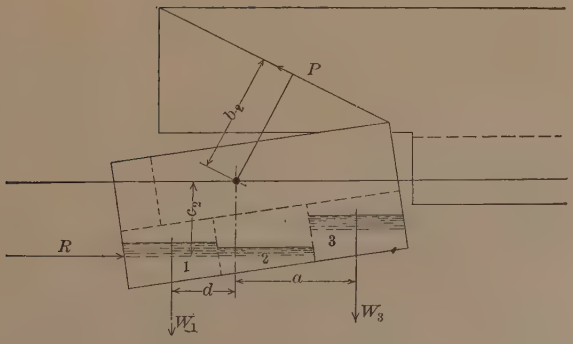


FIG. 5.

Mr. Hansson. While sinking, the pontoon was held in position by hawsers from the after end of the pontoon side walls to the forward end of the main side walls. When the pontoon is floating level, as in Fig. 3, there is a tipping moment, consisting of the pull in the hawser multiplied by its distance to the center of the water line plus the force of the current multiplied by the distance to the center of the water plane. This moment is then counterbalanced by an excess of water in Compartments 3 of the pontoon.

$$P\,b + R\,c = W_3\,a.$$

Now, suppose that the pontoons start heeling, as in Fig. 4. The effect is that while P and R remain practically the same, their leverages grow smaller. The moment, W_3a, may be considered constant for the small angles within which the heeling takes place. There is, then, an unbalanced moment, $W_3a - (Pb_1 + Rc_1)$, tending to heel the pontoon, which increases with the angle of heel until finally balanced by the stability moment. To right the pontoon, the operator lets water into Compartments 1. As the pontoon rights itself, the moment, $W_3a - (Pb_1 + Rc_1)$, grows smaller, and becomes zero when the pontoon is level. There is now, however, an unbalanced moment, W_1d (Fig. 5), caused by the extra water let into Compartments 1, which heels the pontoon the other way. This moment is now being augmented by the moment, $Pb_2 + Rc_2 - W_3a$, which increases as the pontoon heels, until it is finally balanced by the stability moment. This, of course, will repeat itself during the submergence of the pontoon.

Mr. Cox. Leonard M. Cox, M. Am. Soc. C. E. (by letter).—Mr. Bellinger's discussion forms a substantial addition to available data on the cost of self-docking operations and the annual maintenance expense of the larger steel floating docks. His figures, based upon actual experience with the Algiers dock covering a period of four years, may, with an added item to cover extraordinary expenses due to accidents, be taken as a basis for future estimates.

As the Algiers dock lies in fresh water, it is not quite clear why the structure should be self-docked at two-year intervals; in fact, even in salt water, experience with similar docks would indicate that, because of protection afforded by marine growths, this interval could be considerably extended. The paint on upper decks and side walls should, if properly applied in the first place, last two years without renewal. As pointed out by Mr. Bellinger, the paint in the neighborhood of the water line is particularly subject to deterioration, and a belt of good width, extending from light to load draft, should be painted at frequent intervals.

Mr. Bellinger's description of the Cunningham tar mixture enables the writer to repair an oversight in the original paper. The pontoon

deck of the *Dewey* dock was painted with the Cunningham mixture with very satisfactory results. This paint was adopted after trials under working conditions. Two large adjoining squares of the deck were selected; one was given three coats of the adopted red lead mixture and the other one coat of the Cunningham tar mixture. These squares were located on the landward end pontoon, in the building basin, and some two hundred men passed over them daily on their way to different parts of the work. At the end of three weeks the red lead was worn through in places, and at the end of six weeks hardly a vestige remained. The Cunningham mixture, at the time the dock sailed for the Philippines, some seven months later, was still in good condition. It is of interest to note that the naval constructor in charge of the dock at Olongapo is so well satisfied with this paint that he has recently given the deck a new coating. It makes a splendid wearing surface, is cheap as to first cost, and can be applied by unskilled labor. After applying this paint on the deck of the *Dewey,* dry Portland cement was lightly sprinkled over the undried surface. It should be stated, by way of caution, that wherever oil or grease spots were not thoroughly removed from the deck, the tar paint flaked off in scales and had to be reapplied. Mr. Cox.

As stated in the paper, it was thought that exposure of plates to the weather for a period of from one to two years, during the construction of the dock, would remove all mill scale. It became necessary, however, to supplement the weathering effect by hammering, scraping, and washing, before paint could be applied. There can be but little doubt that pickling should be specified for plates of future Government floating docks, at least for the under-water body.

Regarding the question of cranes, it may be said that the floating derrick is to the floating dock what the jib crane is to the graving dock. The jib crane is not a part of the graving dock, but an accessory traveling on its own track from dock to dock, to wharf or workshop. In the same manner, the floating derrick may serve for handling ordnance, boilers or engines from ships alongside the sea wall, or serve the floating dock at its moorings. The plan of installing traveling cranes on the side-wall decks of a floating dock, while appearing at times in general plans accompanying bids for proposed docks, is not practical, because of the necessary interference with guys, stacks, and other working parts of the dock itself. It is conceivable, however, that quick-hoisting, light-lift cranes might be installed at intervals along the outboard edge of the side walls with some gain to the dock's efficiency.

It may not be out of place to state here, in regard to the writer's criticism* of the Clark sectional bolted type of floating dock (erroneously referred to as the Clark and Standfield Pola type), that since

* *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 113.

Mr. Cox. the paper was written, a dock of that description has been built and successfully self-docked.*

Mr. Hansson's proposed use of silting pumps for maintaining the proper depth of water under a floating dock, and his reference to the adoption of such pumps as a part of the permanent equipment of the projected dock for Aberdeen, are interesting because of the fact that the idea was proposed by Civil Engineer A. C. Cunningham, U. S. N., in 1902, and subsequently patented by him. The plan, as patented, consists of an arrangement by which the regular operating pumps of the dock may take water through the flooding valves and discharge it through openings in the bottom. These openings are located so as to cover practically the entire pontoon area. Provision is also made for maintaining the dock at the desired level while pumping.

Traveling cranes may be installed on the side walls so as to permit of their being hauled to the forward end of the dock, thus avoiding interference with the ship's lines in docking, yet all smokestacks would have to be placed on overhanging outboard platforms, and deck-houses, companionway hoods, etc., would have to be located so as to allow the crane legs to pass unimpeded. Altogether, it is doubtful if the advantage to be gained would warrant the additional expense and complication.

In view of the interest shown in the subject of floating docks, the description of the *Dewey's* equipment for the voyage to the Philippines—omitted from the original paper for the sake of brevity—makes Mr. Hansson's discussion of particular value. It should be pointed out, however, that the hawse pipes and bill boards for housing anchors were not contemplated in the original design, and that, therefore, they were attached to the outer shell, forming projections which would not only interfere with vessels in making a landing at bow or stern, but, in the course of the expedition, must often have caused annoyance by fouling the towing gear. Future designs should include such provision, built inside the pontoon structure. If the scow-shaped pontoon be adopted, this object could be easily attained, with the added advantage of having the anchors at all times point fair.

A military floating dock should have a permanent towing equipment which, as far as may be possible, should also be of use in the regular mooring and operation of the dock.

Up to the time of the towing of the *Dewey*, the only experience the Government had had in towing floating docks was with the New Orleans dock, which was towed to its destination by the Boston Tow Boat Company for the contractors, who were to deliver the dock at its site. No special towing apparatus whatever was installed on the New Orleans dock. After a careful inspection of the dock, the towboat company made a bridle of one of the mooring chains, chafing planks

* An account of this dock and its self-docking tests appeared in *Engineering*, July 26th, 1907.

were placed on the deck of the bow on which the bridle could work, and the tow started. It arrived at New Orleans without mishap, and without the parting of a line. That this simple arrangement for towing was not a mere reckless proceeding is indicated by the fact that the towing company was to receive no compensation unless it delivered the dock, and, further, that the underwriters, who assumed the entire risk for the dock, approved of the towing arrangements. Mr. Cox.

The towing of the *Dewey* by the Navy was not decided upon until the last moment, and the undertaking was then placed in the hands of seagoing officers. On account of the responsibility of the work and the entire lack of experience with towing operations of this magnitude, it was natural that provision should be made for every emergency that could be conjectured. Everything asked for, that could be installed in the limited time available, was provided, and much of it was not used.

The general equipment desirable for the towing of a floating dock has been described by Mr. Hansson, and much or all of it would be of permanent value in mooring and docking operations.

The history of the towing of the *Dewey* indicates that its frequent breaking adrift was the result of trying to make too much speed under adverse weather conditions and using too short a tow-line. All these mishaps occurred during the first half of the trip, and, as experience was gained and possibilities realized, such occurrences ceased.

In towing a floating dock the best results will be obtained by using the longest possible tow-line that can be managed, and by accommodating the speed to the weather conditions. It is certainly better to make no headway at all, or even some leeway, than to part a tow-line. By using one vessel of sufficient weight and power to do the towing, the strain on the tow-line may be quickly regulated, and maneuvering may be simplified.

The principal criticism to which the *Dewey* has been subjected has to do with the absence of a pointed bow or cut-water. This feature was omitted, on the advice of the members of the firm who towed the New Orleans dock, all of whom accompanied that expedition. Mr. Hansson's observation appears to bear out the judgment of the designers. The scow-shaped pontoon suggested in the original paper seems to be the most feasible solution of the problem of reducing towing resistance, and it is just possible that that shape would result in a better boat.

Mr. Hansson's explanation of the surging of the forward pontoon during the self-docking tests is simple and satisfactory. As the writer is informed that Mr. Hansson was present at the trials of the New Orleans dock, it would have been interesting had he explained the same phenomena observed with that dock when operated as a unit. The overturning moment in that case could not have been augmented by the pull of holding lines.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1065.

REINFORCED CONCRETE PIPE FOR CARRYING WATER UNDER PRESSURE.*

By Chester Wason Smith, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. F. Teichman, J. R. Worcester, Ernst F. Jonson, R. W. Lesley, William Gavin Taylor, Thomas H. Wiggin, and Chester Wason Smith.

This paper describes the construction of about 6 000 ft. of reinforced concrete pipe intended to carry water under pressure; and also gives the results attained and some figures as to the cost. The work was in charge of the writer.

History and Conditions.

A prominent feature of the Salt River project, in process of construction in Arizona by the United States Reclamation Service, is a canal 19 miles long, having a capacity of 250 cu. ft. per sec. This canal is to furnish power to build the Roosevelt Dam, run the cement mill furnishing the cement therefor, and, later, pump water for additional irrigation in the vicinity of Phœnix.

At Livingston, about 6 miles below the canal intake, the canal crosses Pinto Creek, at that point nearly ½ mile wide, and about 25 ft. below the canal grade.

* Presented at the meeting of October 2d, 1907.

The crossing of Cottonwood Cañon is 2½ miles above the Roosevelt Dam, which is at the lower end of the canal. The crossing is 250 ft. wide on the bottom and 75 ft. below the canal grade.

Pinto Creek has a water-shed of 190 sq. miles. At the canal crossing its grade is about 1%, and the material there is sand with a little gravel and small cobbles.

At the Cottonwood Crossing the cañon has a water-shed of about 4 sq. miles. The grade of the creek is about 4%, and the material is boulders and gravel.

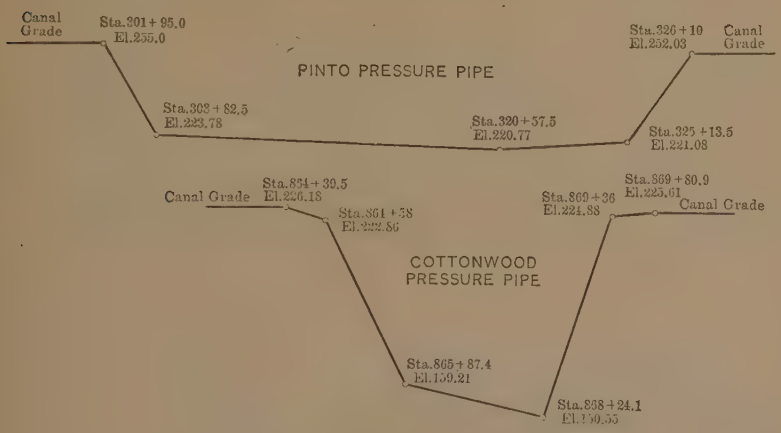


FIG. 1.

Both creeks are dry nearly all the time, except for a small underflow; but they are subject to occasional large and sudden run-offs, characteristic of the flow from mountainous water-sheds which lack vegetation.

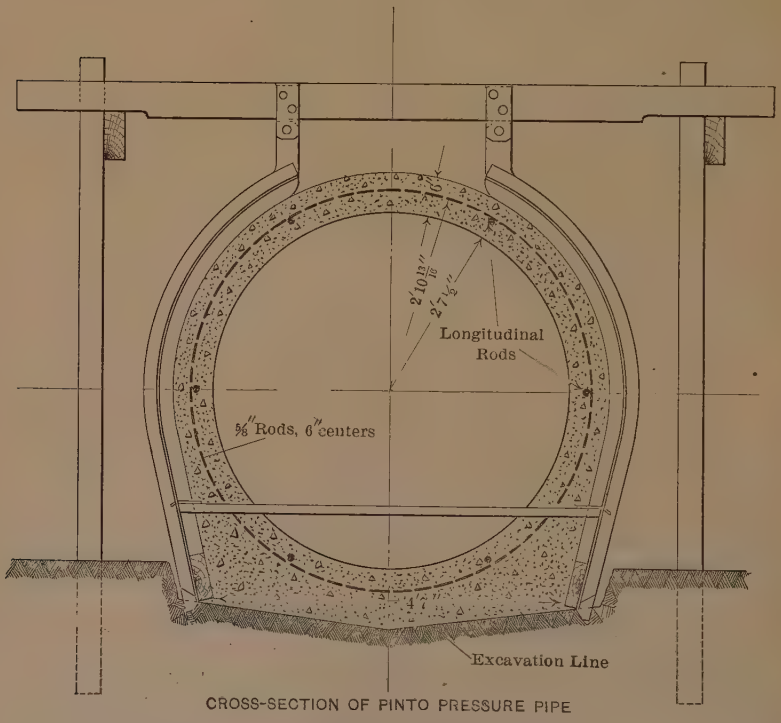
It was decided to make each crossing with two lines of reinforced concrete pipe, circular inside, and 5 ft. 3 in. in diameter, giving the Pinto Crossing an effective grade of 2.97 ft. in 2 415 ft., and the Cottonwood Crossing 0.57 ft. in 541.4 ft. The pipes are buried under the creek, their tops being from 2 to 5 ft. below its bed.

Fig. 1 shows the profiles of the pipes at each crossing, the water level in the canal at full capacity being 5½ ft. above the canal grades noted.

Fig. 2 is a cross-section of the Pinto pipe. The concrete in the Cottonwood pipe is 7 in. thick; there are ten longitudinal rods, and the spacing of the rings is 3 in. from center to center.

The reinforcement, longitudinal and transverse, consists of $\frac{5}{8}$-in. steel rods, having an ultimate strength of 62 000 lb. and an elastic limit of 30 000 lb. per sq. in.

At Pinto Creek there was excellent sand and gravel at all points in the excavation; the water used in the concrete for the first line of pipe was pumped from the trench (the trench intercepting the



CROSS-SECTION OF PINTO PRESSURE PIPE

Fig. 2.

underflow of the creek), and the first pipe was tapped to furnish the water used in building the second one.

At Cottonwood the sand and gravel, and nearly all the water, were hauled about $\frac{3}{4}$ mile from Salt River.

The concrete was composed of 1 part of Portland cement, $2\frac{1}{2}$ parts of sand, and 4 parts of rather fine gravel, and was mixed by

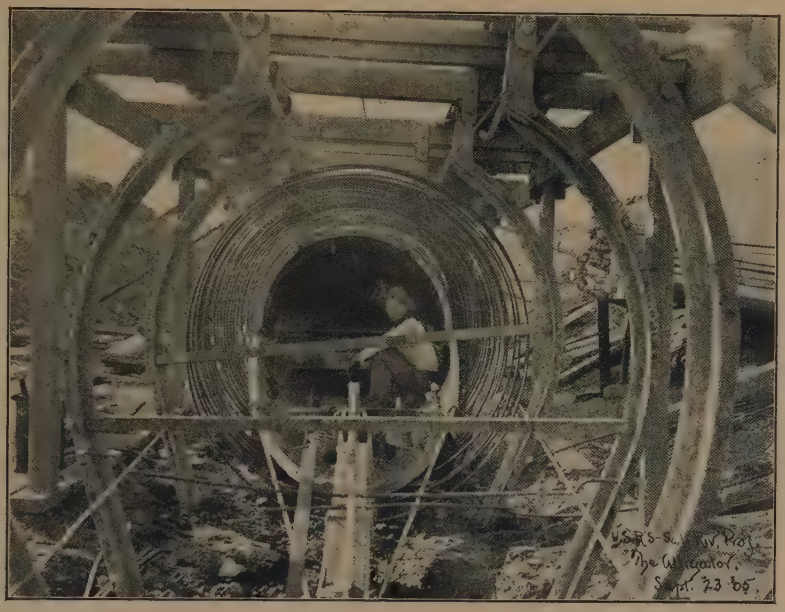


FIG. 1.—THE "ALLIGATOR," USED IN BUILDING REINFORCED CONCRETE PRESSURE PIPE.

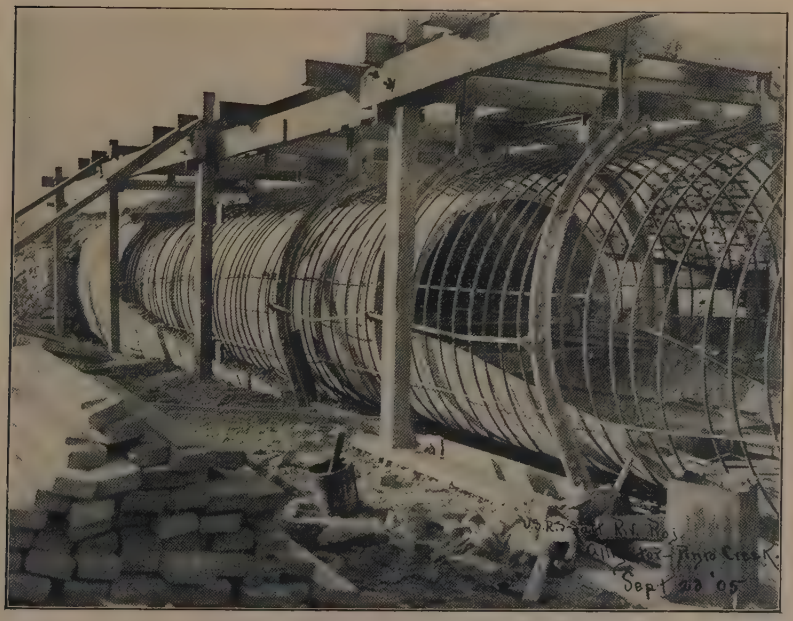


FIG. 2.—THE "ALLIGATOR," USED IN BUILDING REINFORCED CONCRETE PRESSURE PIPE.

hand, quite wet, on platforms of boards about 150 ft. apart along the trench.

The first length of 200 or 300 ft. of pipe was made in June, 1905, with cement purchased from the Portland Cement Company, of Denver, Colo. This was an average Portland cement. A progress of 120 ft. per 24 hours was easily attained with a comparatively raw gang.

Afterward, the cement used was the product of the mill installed and operated by the Reclamation Service to furnish cement for building the Roosevelt Dam.

During the summer and fall of 1905 nearly the entire output of this mill was required by contractors for various other work along the canal, and work on the pipe was frequently interrupted for considerable periods of time.

On account of lack of cement, also on account of several floods, the completion of the first line of pipe across Pinto Creek was delayed until the middle of November, 1905.

As one line would carry all the water needed to furnish power for building the dam, and as power was required as soon as possible, the forms were then moved to Cottonwood, where the two lines were constructed. In March, 1906, the construction of the second line at Pinto was started, but, owing to various delays not connected with the construction, this line was not completed until about August 1st. Much of the time during the construction of both Pinto pipes the temperature was more than 100° fahr. The Cottonwood pipes were constructed entirely in winter, when the temperature was between 35 and 55 degrees.

The cement manufactured at Roosevelt, while excellent as regards strength and soundness, has been extremely slow setting. An average sample would test about as follows:

Fineness: 95% passed the 100-mesh sieve and 76% passed the 200-mesh.

Setting: initial, 4½ hours; final, 12 hours.

Tensile strength: neat, 7 days, 450 lb.; 28 days, 550 lb.; 3 months, 625 lb.; 1 part cement to 3 parts standard sand, 7 days, 100 lb.; 28 days, 200 lb.; 3 months, 260 lb.

A 6-in. concrete cube was made of the materials used in the pipe, except that the proportions were by weight, 1 cement, 2½ sand,

and 4½ gravel, mixed with about the same proportion of water. The batch from which the cube was made was composed of 1 200 g. of cement (of which only 64% passed the 200-mesh sieve), 3 000 g. of sand, 5 400 g. of gravel and 720 g. of water. This cube, after having set for 3 months in water, was crushed at 59 180 lb., or 1 644 lb. per sq. in. This rather low result was due partly to the coarseness of the cement, and partly to the fact that such a wet concrete was slow in attaining its strength.*

With such cement, and 70 lin. ft. of upper stationary plates, it was found that not more than 70 ft. of pipe could be made in 24 hours without getting into difficulties when the plates were removed, as patches of concrete would fall out or peel off with them. With cooler weather, the difficulties increased; accordingly, on the completion of the first line across Pinto Creek, the idea of continuous work was abandoned, and work on the Cottonwood lines and the remaining Pinto line was done with one 8-hour shift per day.

On the Cottonwood lines, built in December and January, it was found that only 24 ft. of pipe per day could be built, thus allowing 3 days for the concrete to harden before removing the plates.

The second Pinto line, constructed in warmer weather, was built at the rate of 40 ft. per day.

Movable Form.

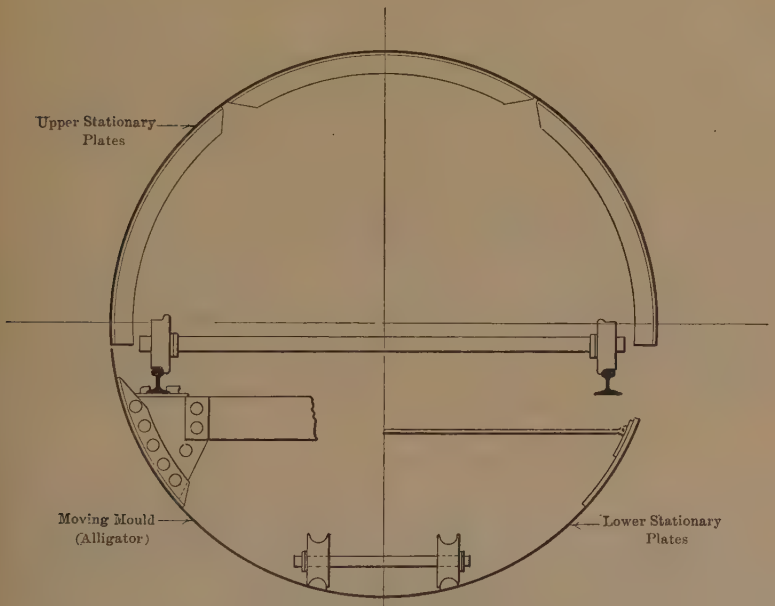
It was considered desirable to work continuously on the pipe, with three 8-hour shifts, in order to avoid, as far as possible, transverse joints, and consequent probable opening between the work of different days.

To facilitate continuous work, a movable form was designed and introduced on the work by F. Teichman, M. Am. Soc. C. E., Designing Engineer in the Reclamation Service.

Briefly described, the form, as shown in Figs. 3 and 4, and the photographs on Plate VIII, is as follows: A steel semi-cylinder, called "the alligator," forms the inside of the lower half of the pipe; this piece is pulled along by a cable from a horse-power whim

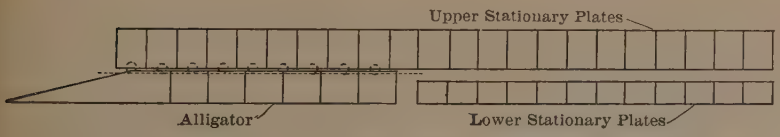
* Cubes since made from batches of concrete being put into the Roosevelt Dam have been crushed, at 3 months, as high as from 89 000 to 103 000 lb. per sq. in. This concrete was composed of 1 part of cement, 2½ parts of sand made by crushing dolomite limestone, and 4 parts of crushed sandstone, mixed to the same degree of wetness.

in the trench ahead. It is kept to line and grade by a steering apparatus, extending about 8 ft. in front of its nose, and either rolling or sliding on a light wooden track previously laid. The inner form for the upper half of the pipe consists of steel semi-



MOVABLE FORM
FOR BUILDING REINFORCED CONCRETE PRESSURE PIPE

FIG. 3.



PART OF MOVABLE FORM FOR BUILDING REINFORCED CONCRETE PRESSURE PIPE

FIG. 4.

cylinders in 2-ft. lengths, each in three pieces, that is, hinged at two points, to facilitate moving and erection.

These upper stationary plates are bolted together, end to end, making a continuous form, from the front end, where concrete is

going in, to the rear end, where the concrete has set sufficiently to permit their removal; they are supported by rollers on a track which is part of the alligator, the alligator thus rolling out from under the upper stationary plates. Immediately behind the alligator are introduced lower stationary plates in 2-ft. lengths, one plate being inserted as often as a length of 2 ft. of invert is exposed. On the withdrawal of the alligator, an upper stationary plate is thus supported by the plate ahead on the alligator and a plate behind on a lower plate, until the insertion of its lower plate. Upper and lower plates are removed at the rear end and sent ahead on a small truck hauled back and forth with a rope, on a track in the bottom of the lower plates and alligator.

The outside lagging was of 2½-in. lumber in narrow pieces, about 5½ ft. long, laid on the same slope as the nose of the alligator (1 in 4) between iron ribs hung from a wooden superstructure. This superstructure also carried the runways for wheeling out the concrete.

Construction.

The rings were bent to the desired circumference by a small bending machine. The ends were not upset or welded, but merely lapped about 15 in. and tied with baling wire; they were also wired at each crossing with each longitudinal rod, in order to hold them in place while the concrete was being put in.

The concrete was made with fine gravel, and mixed quite wet, in order to be worked into the narrow space between the reinforcement and the forms completely and easily. It was brought out in wheel-barrows on top of the superstructure and dumped wherever desired along the working face, that is, from the bottom of the pipe at the nose of the alligator, back on the 1 in 4 slope to the top of the pipe at the third or fourth upper stationary plate. One or two small chutes were found desirable in connection with the dumping.

Two men on each side, provided with small wooden paddles, churned the concrete to make certain that all voids were filled, and to prevent any nesting of the gravel. These men also introduced the outside lagging, that operation and the moving ahead of the alligator being regulated so that the working face was from

FIG. 1.—THE CONSTRUCTION OF THE PINTO PRESSURE PIPE.

FIG. 2.—THE CONSTRUCTION OF THE COTTONWOOD PRESSURE PIPE.
(The inclines were built with wooden forms, concrete being run down in chutes from above.)

0 to 8 or 10 in. below the forms. On stopping work for the night, or if the continuous operation was interrupted for any reason, the blocking off was done at right angles to the axis of the pipe. This was done by stuffing sand bags between the inner and outer forms. This was more convenient than the use of a wooden stop, and afforded a rougher surface with which to bond when work was resumed.

The pipe was wet down on the outside for about a week after completion, and, as far as possible, the back-filling was kept up to the watering.

Some smoothing and pointing was necessary inside the pipe, and one or two brush coats of neat Portland-cement grout were applied; but no plaster coat was put on, nor was any water-proofing material used.

Difficulties.

It was difficult to overcome the tendency of the alligator to twist and travel off line or grade, and the steering apparatus was designed to prevent this, it not having been a part of the original form. It did much toward correcting the eccentricities in the travel of the alligator, but, at best, much depended upon constant watchfulness, and it was found necessary to stop work once in 8 or 10 days in order to level and straighten it out.

It is very doubtful if a uniform thickness could have been attained, even had an attempt been made to shift and adjust the outside forms and the reinforcement to correspond with the movements of the alligator. With this form, the minimum thickness will be at least 1½ in. less than the average thickness.

Considerable trouble was caused by the concrete peeling off with the upper stationary plates; particularly—in fact almost solely—with the middle segment of the plate. Large patches, often an inch or more in depth, would come off with the plate.

To overcome this, various schemes were tried; soft soap or crude oil was applied to the plates immediately before covering them with concrete; particular care was taken to allow no spatterings of concrete to accumulate on the plate and dry out in advance of covering it with the mass of concrete; variations in the quantities of water in the concrete were also tried. Experience seemed to

show that the only satisfactory remedy was to allow the concrete to set more thoroughly before removing the plates; if given sufficient time, a perfect surface was obtained. No special precautions were taken to guard against transverse joints between the work of different days, but, when an end was to be left for more than one day before resuming work, extra longitudinal rods were inserted, extending about 2 ft. into the work on each side of the joint.

Results.

Transverse Joints.—On the first Pinto line, where there were few interruptions to continuous work, and where a portion of it was built in comparatively cool weather, there were very few transverse joints worthy of mention, and none requiring any treatment. On the two Cottonwood lines, built in the coldest weather, none has been observed.

On the second Pinto line, built entirely during the hottest weather, with about fifty interruptions to continuous work, and first filled in cold weather, it was to be expected that the transverse joints would be larger in number and size. At about forty of them there was a perceptible crack; of the forty cracks about twenty-five were $\frac{1}{16}$ in. or more in width, and were repaired, some of them before the pipe was first filled, and the remainder afterward. In general, they were more frequent and pronounced in one or two sections where the watering of the pipe was discontinued before the back-filling was done.

Longitudinal Cracks.—When the pipes were put in service, some longitudinal cracks, from $\frac{1}{16}$ to $\frac{3}{16}$ in. in width, developed (see leakage tests in Table 2), and at first various possible causes or contributary causes were assigned to them:

First, they were ascribed to the water hammer, which was considerable when the pipes were being filled.

Second, it was thought that the excavation of the trench for the second line of pipe (10 ft. from center to center of the first line) threw an eccentric load on the first pipe. This undoubtedly had an effect on the cracks in the first Pinto pipe, and, as soon as it was observed, the remainder of the second pipe was moved 10 ft. farther away.

Third, the cracks being invariably in the top of the pipe, and within a few inches of the center, it was thought that possibly the laps in the reinforcing rings had been put in on that line, and that the rods, or some of them, had parted from the concrete. Enough evidence was at hand, however, to show that the laps had been properly staggered. If such had been the cause, a longitudinal crack developing in a few hours to 200 or 300 ft. long, would have resulted in the total failure of that portion of the pipe.

The true reason is undoubtedly as follows: The concrete shrank in the process of setting; this was resisted by the steel rings, thus producing a condition in which the steel was in compression and the concrete in tension. Therefore, on filling the pipe, the concrete took the entire load until it failed, and then the steel took the load.

That the cracks were invariably in the top of the pipe was probably due to the fact that the lower third of the pipe was in sand kept wet by the underflow; in effect, this lower part set under water, and the shrinkage was relatively small.

A careful inspection of the inside of the pipe showed that, wherever the crack occurred, the single large crack was the only one; and, at points where there was no large crack, the upper two-thirds of the inside surface showed a network of very minute cracks. These were from 1 to 4 or 5 in. apart, and generally so fine that they could have had but a very insignificant effect on the leakage, and, in fact, they were only detected by the presence of a minute line of the finest sediment which was deposited at the crack without passing through.

The writer is of the opinion that the minute cracks existed previous to the filling of the pipe, consequently, that at those places the steel carried the load immediately upon its application; however, there is no way to verify this. Repairs were made (on the inside of the pipe) in the following manner: each crack was cut out to a depth of about 2 in. and as narrow as possible, 1 in. of oakum was then caulked in tightly, and over that the joint was pointed with stiff mortar. In addition, grout was run into the crack from the outside.

Several observations have been made for the purpose of determining the value of n, in Kutter's formula, and, while the en-

tire series contemplated has not been made, the observations on the Cottonwood pipes, thus far, show that $n = 0.012$, approximately.

In considering the results of the leakage tests, it should be borne in mind that Salt River carries considerable sediment. Observations at Roosevelt have shown that the quantity varies from a mere trace up to 4 or 5 per cent. Practically, the water is never without some sediment, but the maximum occurs only during the highest floods; the average would be between one-fourth and one-half of 1 per cent.

Between Roosevelt and the canal intake, considerable quantities of the sediment, at ordinary stages of the river, settle in the flat reaches, so that at the intake, while the maximum quantity of sediment is not more than at Roosevelt, the average is undoubtedly greater.

At several points along the canal there are mud boxes from which more or less sediment is occasionally sluiced, but the canal carries a larger percentage of sediment than the river, except when the latter is in flood.

Cost.

The figures in Table 1, as to the cost of two sections of the second Pinto pipe, show the labor cost only, and do not include engineering, first cost of forms, cement, reinforcement, or grading.

TABLE 1.—Cost of Pipe.

	714 lin. ft. May, 1906.	1 009 lin. ft. July, 1906.
Laying track for steering alligator	$71.48	$43.98
Moving and erecting superstructure	299.94	358.44
Moving plates	202.50	252.44
Repairs to alligator	58.50	2.50
Bending rings	32.87	59.87
Placing reinforcement	126.94	138.13
Mixing and placing concrete	709.68	949.74
Watering	45.00	78.27
Pointing up and brush-coating inside	96.50	117.37
Blacksmith's work	30.00	25.00
Whim	23.87	23.75
Screening and hauling sand and gravel	183.13	300.00
Total	$1 880.41	$2 355.49
Barrels of cement used	466½	627
Number of days' work	18	26
Labor cost per linear foot of pipe	$2.63	$2.33
Labor cost per cubic yard of concrete	5.93	5.25

A gang consisted of a foreman at $175 per month, a sub-foreman at $3.50 per day, and the following laborers at $2.50 per day: one bending the reinforcement rings; two placing the reinforcement; four taking down, moving and erecting the stationary plates; four placing the concrete and outside lagging; two wheeling concrete; six mixing concrete; one wheeling sand and gravel; one watering the finished pipe; four laying track for the steering apparatus, moving the superstructure and hangers, mixing-boards, runways. etc.; one pointing and finishing inside the pipe; and one on the whim and doing miscellaneous work. The labor was principally Mexican, and only fairly efficient.

Results of Leakage Tests.

The leakage in the pipes was measured by observing the water level in them, all water being shut off. The head mentioned in the tables is the elevation of the water level above the lowest point in the pipe. The periods are consecutive in each case, that is, on January 25th the periods were during 2 consecutive hours, and on January 26th, during 6 consecutive hours, etc.

From February 26th to March 15th the pipe was in use a large part of the time. Various longitudinal cracks developed from time to time and were repaired. The next opportunity to measure leakage was from March 15th to 22d, after all the cracks had been repaired, since which time no more have appeared.

During the construction of the south line of the Pinto pipe, one lot of inferior cement, containing free lime, was received on the work, and a portion of it was used before being discovered. The result was a slower setting concrete, and at several places the upper stationary plates were removed too soon, allowing the concrete inside the reinforcement to settle and part from the outside.

Of these places, the worst one was repaired before the test of December 14th was made, but several minor ones, as well as most of the transverse joints, were not repaired until after that test.

TABLE 2.—Leakage Tests.—Pinto Pressure Pipe.—North Line.

This pipe was built between June and November, 1905, and first filled with water on January 25th, 1906.

Date.	Duration of test. Hours.	Minutes.	Leakage, in gallons.	Gallons per hour.	Average head.
Jan. 25, 1906	0	30	1 225	2 450	30.2 to 26.5
" 25, 1906	0	30	1 225	2 450	
" 25, 1906	0	30	1 130	2 260	
" 25, 1906	0	30	992	1 984	
Jan. 26, 1906	1	00	.....	2 205	30.5 to 22.4
" 26, 1906	1	00	.....	1 960	
" 26, 1906	1	00	.....	1 837	
" 26, 1906	1	00	.....	1 592	
" 26, 1906	1	00	.....	1 102	
" 26, 1906	1	00	.....	1 225	
(The pipe was full of water from January 26th to February 17th.)					
Feb. 17, 1906	1	00	.....	300	21.5
Feb. 22, 1906	1	00	.....	183	27.9
(On February 26th a longitudinal crack, 40 ft. long, developed.)					
Feb. 26, 1906	1	15	7 035	5 628	26.0 to 20.2
" 26, 1906	12	15	4 490	367	20.2 to 16.5
" 26, 1906	1	00	.....	86	16.5
(This test shows the effect of a crack, and the effect of a small reduction of head.)					
Mar. 15, 1906, 8:15 A. M., to Mar. 22, 1906, 6:30 A. M.	2	30	375	150	26.9
	7	00	350	50	26.6
	14	30	1 650	114	26.3
	4	15	475	112	25.9
	5	00	420	84	25.6
	16	45	1 575	94	24.7
	6	45	460	68	23.9
	19	00	1 740	92	23.0
	6	15	375	60	22.1
	13	30	1 610	119	21.3
	11	30	715	62	20.4
	11	15	1 100	98	19.6
	13	00	730	56	18.9
	13	30	730	54	18.3
	11	00	460	42	17.8
	10	30	375	36	17.4
Dec. 19, 1906	16	00	9 893	618	30.5 to 20.9
	1	00		550	
	1	00		458	
	1	00		458	
	1	00		458	
Jan. 30-31, 1907	24	00	92	3.8	30.5
(This test shows the effect of a run of muddy water.)					

TABLE 2.—(*Continued*).—LEAKAGE TESTS.—PINTO PRESSURE PIPE.—SOUTH LINE.

Built March to August, 1906, and first filled on December 15th, 1906.

Date.	Duration of test. Hours.	Minutes.	Leakage, in gallons.	Gallons per hour.	Average head.
Dec. 15, 1906	1	00	18 320	18 320	30.5 to 15.5
	1	00	3 664	3 664	15.5 to 12.5
	1	00	489	489	12.5 to 11.6
	1	00	366	366	
	1	00	290	290	
Jan. 26, 1907	2	00	3 191	1 595	30.5 to 27.9
	2	00	3 100	1 550	27.9 to 25.4
	12	00	13 648	1 137	25.4 to 14.2
	2	00	641	320	14.2 to 13.7
Jan. 29-30, 1907	1	00	2 198	2 198	30.5 to 28.7
	1	00	2 107	2 107	28.7 to 27.0
	1	00	1 832	1 832	27.0 to 25.5
	1	00	1 832	1 832	25.5 to 24.0
	1	00	1 649	1 649	24.0 to 22.7
	1	00	1 465	1 465	22.7 to 21.5
	18	00	9 984	555	21.5 to 13.3

TABLE 2.—(*Continued*).—LEAKAGE TESTS.—COTTONWOOD PRESSURE PIPE.—SOUTH LINE.

Built between December 25th, 1905, and January 26th, 1906, and first filled on March 9th, 1906.

Date.	Duration of test. Hours.	Minutes.	Leakage, in gallons.	Gallons per hour.	Average head.
Mar. 9, 1906	0	30	535	1 070	38.4 to 34.3
" 9, 1906	2	30	1 452	580	
" 9, 1906	1	00	.....	473	
" 9, 1906	1	00	.....	443	
" 9, 1906	1	00	.....	370	
Mar. 10, 1906	0	30	1 475	2 950	49.0
Mar. 27, 1906	2	30	38 100	15 200	74 to 18
Sept. 22, 1906	7	00	0	0	74

The south line was the first one constructed at Cottonwood. Some time in April the pipe failed by blowing out a hole about 4 ft. square in its top. This was at the point where the first roof plates were removed, and would indicate that they were removed too soon, allowing the concrete inside the reinforcement to settle a little and part from the reinforcement and the outside concrete. The hole was cut out to solid work, and patched.

TABLE 2.—(*Continued*).—Leakage Tests.—Cottonwood Pipe.—North Line.

Built January 8th to 20th, and first filled on March 9th, 1906.

Date.	Duration of test. Hours.	Minutes.	Leakage, in gallons.	Gallons per hour.	Average head.
Mar. 9, 1906	0	40	167	251	24.0 to 22.5
	2	30	591	237	
	1	00	.....	177	
	1	00	.....	121	
	1	00	.....	98	
Mar. 10, 1906	0	30	3 540	7 080	69.0
	2	30	12 500	5 000	60.0
	(8-ft. longitudinal crack opened.)				
Mar. 23, 1906	2	15	4 525	2 010	60
	0	30	1 048	2 097	73
Mar. 27, 1906	2	20	9 000	3 860	74.0 to 60.7
Mar. 28, 1906	5	45	.....	1 170	73 to 63
Mar. 29, 1906	18	45	.....	800	58
Apr. 16-17, 1906	26	00	.....	39	74
May 24-26, 1906	55	00	863	16	74
May 28-29, 1906	48	00	000	00	74
Sept. 22, 1906	24	00	000	00	74

Between March 10th and 27th various short longitudinal cracks developed and were repaired. The pipe was in service intermittently. Since the end of March the pipe has been almost constantly in service.

Conclusions.

The very slow setting cement used on this work rendered it impracticable to work continuously on account of the excessive number of stationary plates which would have been required.

With a cement setting in average time, the continuous process would certainly be practicable and advisable; and a machine similar to the one described herein, or designed to accomplish similar results, would probably be the best solution of the question of forms.

The necessity of continuous work may be said to vary with the temperature at which the pipe is constructed; at low temperatures it would be almost a matter of indifference; even at high temperatures it is probable that continuous work would not be absolutely necessary, because a method of making connections which would result in a nearly water-proof joint could probably be devised.

There are many engineering works, in process of construction, or projected, for conveying water under pressure, where similar reinforced concrete pipe could be used, and would be, were engineers satisfied as to its first cost, durability and reliability.

The question may be asked, what is the maximum head for which this kind of construction could be used? The precedent for such construction, as well as the literature on the subject, are both believed to be quite meager, though the writer has had little opportunity for searching engineering literature. Attention might be called to a paper* by J. H. Quinton, M. Am. Soc. C. E., entitled "Experiments on Steel Concrete Pipes on a Working Scale."

The excessive amounts of leakage reported in that paper would seem to have been due to the use of a dry mixture, and the writer would take decided exception to the conclusion (expressed on page 55), that "reliance for impermeability must be placed on the plastering rather than on the material of the pipe." Reliance should be placed upon a wet mixture of the material of the pipe.

There is no doubt that the first cost would be less than for either cast-iron or riveted-steel pipe, and would be little if any in excess of wood-stave pipe.

As regards durability, the question is as to the relative durability of the metal in the various kinds of pipe, disregarding for the moment the life of the wood staves.

The reinforced concrete pipe should be far less likely to be destroyed by electrolytic action than either cast-iron or steel pipe. The reinforcement receives a coating of cement which stays with it when the surrounding concrete is cracked or broken away, and the question becomes one of the relative efficacy of the cement coating on one hand, and the asphalt coating of the metal pipe on the other, remembering that the cement coating is absolutely protected against abrasion, and also enclosed in a manner which renders inspection difficult and incomplete.

It may be observed here that the metal in the reinforced concrete pipe possesses the same advantage over the metal in the riveted-steel pipe which has been advanced in favor of the bands on wood-stave pipe; *i. e.*, of form, being a rod with a diameter considerably greater than the thickness of the riveted-steel pipe. In

* Water Supply and Irrigation Paper No. 143 of the United States Geological Survey.

other words, given a certain quantity of steel per linear foot of pipe (enough to carry the load), the most durable form would be that with the least amount of exposed surface.

The problem of inspection might be solved by cutting into the steel occasionally at suspected points, such as at some crack or leak. The reinforcement in the concrete pipe should have a much longer life than the bands on the wood-stave pipe, though, on the other hand, the latter can be readily inspected and renewed.

Of course, as to the relative permanence of wood and concrete, there can be no question, even admitting the claims of the most ardent wood-stave advocates.

The reliability, or safety against sudden failure under a high pressure, seems to the writer to be simply a matter of correct design (to be determined, doubtless, by experiment), and the character of the inspection of the materials and construction. It is not necessary to state that the inspection should be absolutely first-class. It would seem that in correct design the relative quantities of concrete and steel should be such that the concrete in shrinking cannot compress the steel, but will crack; then, with nearly perfect adhesion of the two, there should be many cracks, but so minute that they will readily silt up.

Whether the concrete cracked at few places or many would depend upon the quality of the concrete, the relative quantities of concrete and steel, and the degree of perfection of the adhesion. In this there is a field for experiment, in order to determine the strength of adhesion of concrete and steel, the best form of reinforcing rod, and the amount of shrinkage during the setting of various mixtures of concrete.

It will undoubtedly be found that there is a minimum advisable spacing for the reinforcing rings, on account of their tendency to form a plane of cleavage and thus separate the concrete into an inner and an outer shell; also to insure the complete filling of the entire space with the concrete. For a large quantity of steel in the section, the arrangement might be in the form of two or more rows of rings staggered.

This paper is presented, not as pretending to solve the problem, but with the hope that the governing considerations have been stated correctly, and that the suggestions will be of some value.

In conclusion, the writer wishes to acknowledge valuable assistance derived, during the construction of the pipe and the repair of the cracks, from a free discussion of the various problems with Louis C. Hill, M. Am. Soc. C. E., Supervising Engineer; F. Teichman, M. Am. Soc. C. E., Designing Engineer, and Mr. A. L. Harris, Assistant Engineer; also the exceptionally conscientious work of Mr. A. P. Cox, Foreman.

DISCUSSION.

Mr. Teichman. F. TEICHMAN, M. AM. SOC. C. E. (by letter).—To make good concrete pipe, and to make it at a low cost, three things are required: experience, care, and a good apparatus for moulding the pipe. As to the mould used in building the concrete pipes of the Salt River Project: it gave satisfactory results, on the whole, but by close observation of the apparatus in the field, and of the pipe made by the apparatus, the writer (who designed the mould) arrived at the following conclusions:

1.—For a pipe large enough in diameter to allow a man to do the dismantling of the inner mould, no parts of this mould should be pulled along on the inner face of the pipe (as was the alligator). In preference, the dismantled parts of the rear should be forwarded to the front, and should be erected rigidly there. The three reasons against a sliding mould are: The continuous or intermittent pulling ahead of the inner mould or parts thereof (the alligator) is likely to disturb the support of the concrete while setting, thus establishing a weakness in the concrete, or a defective union between the reinforcement and the concrete. Parts of the concrete while setting may adhere to the moving mould, thus making a face that is marred, defective in appearance, and in compactness and density of concrete. The movement of the mould is not under easy control as to alignment, and it is impossible to make, at will, angles in the alignment of the pipe without interrupting the continuity of the work.

2.—The ribs for the lagging of the outer mould should not be suspended from a framework, but should be supported preferably on sills resting on the ground. The support on sills gives greater rigidity to the outer mould than the suspension from a framework.

3.—The wheeling planks should not be supported by the same structure which supports the mould, otherwise the jars occasioned by the loaded wheel-barrows or cars may be felt in the mould, and may injure the concrete work.

4.—The inner and outer moulds should be erected on the same base —sills laid on the ground—thus assuring their concentricity. Concentricity makes possible the reduction of the thickness of the pipe to a minimum, and a pipe is cheaper and better, the thinner its walls are, provided the reinforcement is properly embedded and the concrete of the wall is strong enough as a beam to support the inner pressure between two adjoining reinforcement rings.

5.—Inside the inner mould there should be a track, with a car, to carry to the forward end of the mould the plates dismantled at the rear. This track should rest on rollers, placed at intervals near the bottom of the inner mould. A track, thus supported, can be pushed ahead at will so as to be always in a convenient position, for both the rear and front plate men.

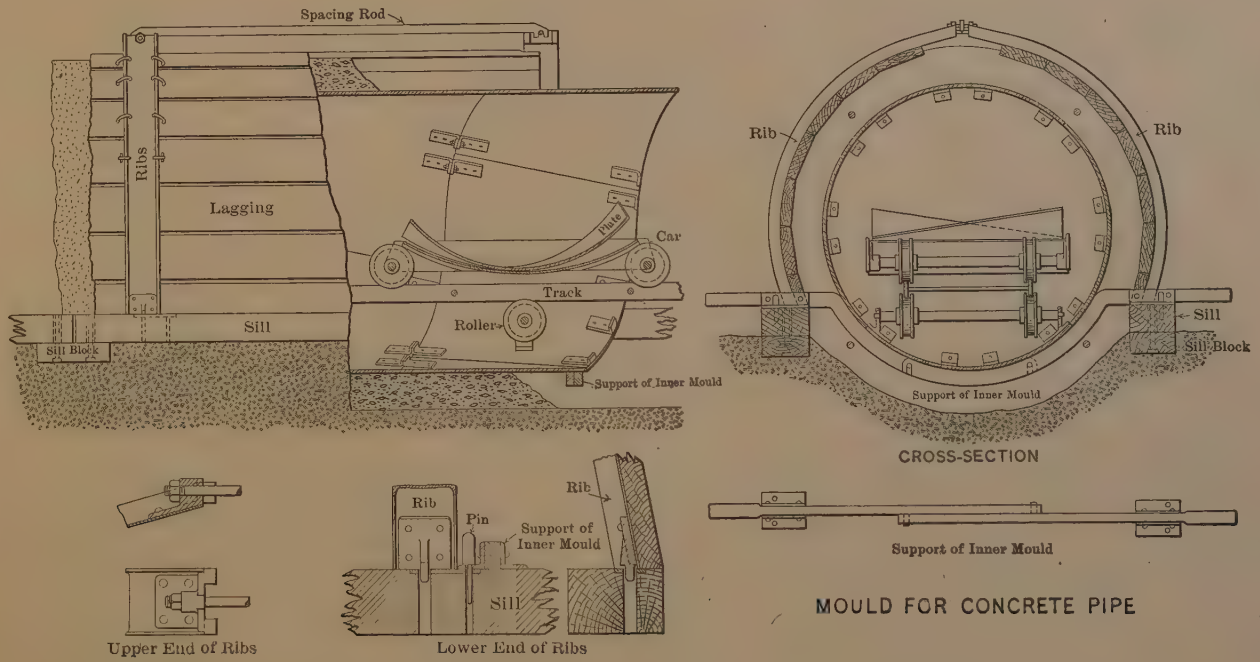

Spacing Rod
Ribs
Lagging
Plate
Car
Track
Roller
Sill
Sill Block
Support of Inner Mould
Rib
Rib
Sill
Sill Block
Support of Inner Mould
CROSS-SECTION
Rib
Pin
Support of Inner Mould
Sill
Rib
Support of Inner Mould
MOULD FOR CONCRETE PIPE
Upper End of Ribs
Lower End of Ribs

FIG. 5.

Mr. Teichman. 6.—Instead of arranging the plates of the inner mould in individual courses, square to the axis of the pipe, it is preferable to arrange them in a continuous spiral, in such a manner that their joints, which are square to the spiral, will always be staggered, *i. e.*, strike the plates of the adjoining spiral courses in their middle. For pipe of ordinary diameters, this will give stiffness to the inner mould, even without the use of stiffening angle iron, or brace rods. The spiral plates for a cylindrical pipe mould are as easily made as those for individual courses, but are more easily dismantled and erected than the latter.

7.—The ribs of the outer mould should be arranged so that the two halves of one rib may be spread more or less at the top as well as at the bottom (with the sills), in order to permit some variation in the thickness of the concrete. The rib halves must be removable before any lagging inside the ribs is removed.

On the basis of these principles, the writer has designed a mould for ordinary pipes of small diameter, as illustrated in Fig. 5. The supports for the inner mould, of course, are in use only ahead of the concrete work, and, as the concrete approaches them, they are withdrawn (by first spreading the two halves) and moved to the front. While in use, these supports are pinned to the sills, just as the ribs of the outer mould are. This secures concentricity of both moulds. Any slight change in the direction of the pipe is made by leaving open the spiral joints of the inner mould at one side and covering the gap with light iron or tar-paper. At such changes of direction, special sills and lagging, cut to special lengths, are required for the outer mould.

The inner track will accommodate itself to slight lateral changes by connecting the different lengths of track by bolts in slotted holes.

The radial dimension of the supports of the inner mould is small enough so that these supports are always inside the longitudinal reinforcing rods, and the length of the arch of these supports is such that the supports may be withdrawn without interfering with these reinforcing rods, if they are spaced in the usual manner.

Mr. Worcester. J. R. Worcester, M. Am. Soc. C. E. (by letter).—The author's experience with these pressure pipes is particularly interesting, in the light it sheds upon the question of the amount of longitudinal reinforcement necessary to prevent transverse cracks in long, straight structures.

It has been held by some authorities that longitudinal steel to the amount of half of 1% of the area of the concrete is necessary, and it is generally conceded that the proportion is wholly efficient. In underground pipes and tunnels, many have ventured to use considerably less than this, but the determination of the precise quantity has not yet been made, and, in the mean time, all light shed on the subject is very valuable.

Mr. Worcester.

It seems that, in the case of the Pinto pipe, the proportion of longitudinal steel to concrete in the portion of the section with cylindrical exterior was about 0.0014. That in one of the pipes of this line there should have been 40 transverse cracks in 2 400 ft., or one to each 60 ft., would not seem surprising, considering the low percentage of steel; and it would seem to be due to an unusual combination of favoring conditions that the other line showed so few.

In the Cottonwood lines, where the ratio of steel to concrete was about 0.002, it would be interesting to know how much of the immunity from cracks was due to the conditions under which the pipes were laid, and how much to the greater percentage of steel.

Mr. Jonson.

ERNST F. JONSON, ASSOC. M. AM. SOC. C. E. (by letter).—There is one point in Mr. Smith's interesting and timely paper which stands out as a possible problem to be solved in future attempts to build concrete pipes, namely, the fact that these pipes cracked both longitudinally and transversely.

If the water of Salt River had not contained considerable sediment, the cracking of the pipes might have been a serious matter. In many cases it might be necessary to design concrete pipes in such a way that they would not crack, partly on account of the excessive leakage due to cracks, and partly on account of the uncertainty about the durability of the reinforcement where it is intersected by cracks.

Longitudinal cracks are probably due to two principal causes:

1.—Shrinkage in the concrete, due to dryness while setting: This shrinkage causes an initial tension in the concrete, resisted by an initial compression in the steel. This tension is an addition to the stress produced by the hydrostatic pressure, so that the pipe will crack at a lower pressure than if no shrinkage had taken place.

2.—A strain in the reinforcement greater than that corresponding to the ultimate tensile strength of the concrete.

Cracks of the first kind might be prevented by keeping the concrete thoroughly wet; those of the second kind by using a very low unit stress on the reinforcement.

Pipes constructed on the foregoing principle might have walls of considerable thickness: First, because the concrete supplies the required tensile strength more cheaply than steel, when only about one-tenth of the safe tensile strength of the latter can be utilized; second, because the leakage through concrete decreases as the thickness increases; and third, if the concrete is not to crack, there is no objection to a considerable thickness.

The quantity of reinforcement in this case may be greatly reduced, leaving only enough to prevent failure in case the concrete should crack. For this purpose the steel might be strained up to 16 000 lb. per sq. in., or even higher.

Pipes designed on this basis, however, would not be economical for

Mr. Jonson. greater pressures than about 60 lb. per sq. in., which corresponds to a head of about 150 ft. When the concrete becomes too thick, in relation to the diameter of the pipe, any addition to the thickness adds very little to the strength of the pipe, because the stress decreases very rapidly toward the outside.

Transverse cracks, the author states, appeared in those parts of the pipe which were built in warm weather. They were evidently due, therefore, principally to temperature contraction. Such cracks might be avoided by leaving open contraction joints to be filled with grout in cool weather.

With regard to Table 2, the writer would make the following suggestions:

Instead of giving the leakage for the entire pipe, it probably would have been better to give the average leakage per unit of length, thus making possible an immediate comparison between the various pipes.

Instead of giving the head at the lowest point of the pipe, it would have been better to give the average head for the entire filled portion of the pipe. The leakage is a function of this average head more directly than of the head at the lowest point.

It would have been desirable to know the hour, as well as the day, when the tests were made.

It would have been interesting, also, to know the temperature of the water during the tests, as this affects the leakage on account of the variations in viscosity.

An additional column, giving the leakage per unit of length divided by the average head, would also have been of value, as showing more clearly the variation in the permeability of the concrete. The leakage may be assumed to be proportional to the head, except in the case of large cracks.

Mr. Lesley. R. W. Lesley, Assoc. Am. Soc. C. E.—The author does not state the reason for constructing the pipe in the particular manner in which it was built. In Europe, for similar purposes, conduits of similar capacity have been built in sections. This form of construction is by no means new, for the "Bordenave" system was used in the water supply of Venice nearly twenty-five years ago. Recently, the scientific papers seem to be quite full of descriptions of the construction of water systems with concrete pipes built according to some of the methods in use in France.

Mr. Arthur E. Collins, City Engineer of Norwich, England, who visited France to investigate the "Bonna" system of concrete pipe, for pressures up to 300 ft. head, which he was about to use on some 2½ miles of concrete main in his city, reports that he found pipes made under this process at Maison Alfort, in the Department of the Seine, such pipes having a length of 10 ft., an internal diameter of

2 ft. 7 in., and a thickness of wall of 2⅜ in. In Paris, he saw pipes of this system carrying working pressures of 150 ft. head, without oozing or other noticeable defects. A similar report was given on work at Nîmes, where ⅔ mile of pipe had been laid eleven years ago and was entirely satisfactory. Mr. Lesley

There is a high-pressure concrete water pipe of this character at Swansea, England.* The main is 3 600 ft. long and 19.7 in. in diameter, operating under a head of 185 ft. The pipe consists of an inner and an outer reinforcement separated by a sheet-steel tube and all embedded in a 1 : 2 mortar. The inner and outer reinforcements consist of longitudinal bins of cruciform (+) section wound by a spiral bar of the same section wired to them at every intersection. Only the outer reinforcement and the steel tube are considered in calculating the strength of the pipe, the inner reinforcement being considered as simply supporting the mortar.

In the *English Master Builders' Journal,* of October 2d, 1907, comment is also made as to the extent to which pipe consisting of cement with steel reinforcing rivals cast iron, on the Continent. It is stated in this case that more than 200 miles of such pipe were laid in Paris alone by the "Bonna" system. This pipe can be constructed up to 6 ft. in diameter, and is rapidly becoming an important rival of cast iron.

In Venice the pipes were 6 ft. in diameter and were joined to one another. They were made by standing upright a series of iron rings which were surrounded on the inside and outside by sheet iron. Around the rings was wound a spiral (or two or three spirals, according to the pressure to be sustained by the pipe), and this "hoop-skirt" (as one might call it) of circular rings or spirals wound round it then had concrete poured upon it between the two thicknesses of sheet iron. After the concrete had set, the sheet iron was removed. The pipe was then complete, and was laid in the trench in the ordinary way. This pipe was made under the "Bordenave" process, as already stated.

In the United States, also, a water pipe has been made by what is known as the "Phipps" system. Two concentric cylindrical shells of sheet iron are filled with a layer of cement, and another layer of cement is filled in between a removable core and the innermost sheet. These are made upright, the cement being poured between the two shells and also between the inner shell and the core. When the cement is set the core is removed, thus producing a pipe having a cement interior. These pipes have been made up to considerable sizes.

The speaker cannot state the pressures on the Venice pipe. In the articles referred to, it will be noted that pressures up to 300 ft. have been carried. In the case described by the author there may

* *Engineering-Contracting*, of September 18th, 1907.

Mr. Lesley. have been some reason (not mentioned by him) that made it necessary to build the pipe in the way he describes, but the various systems of concrete pipe just referred to are well known in Europe, and have been used for many years under quite high pressures.

Another point in connection with Mr. Smith's paper is the cost of the labor on the work, but, of course, this may be well explained by the distance from civilization and the difficulty of securing good workmen. The question of cement and the difficulties found with some of it may possibly be explained, as a good deal of the material seems to have been made at the Government cement works at the Roosevelt Dam, and, according to the paper, this does not seem to have been quite as satisfactory as that first used, which came from the Denver Cement Company. In view of the fact that Los Angeles, Cal., is also about to erect a cement works of its own, for the production of the material for the new water supply, it is to be hoped that the question of a municipal or Government cement works has been thoroughly studied out by that city, before undertaking this responsibility, not only in justice to the contractors who are to use the cement, but to those people whose homes may be below the dams to be constructed with the untried material.

Mr. Taylor. WILLIAM GAVIN TAYLOR, M. AM. SOC. C. E. (by letter).—In connection with the construction of works for the purification of sewage in Waterbury, Conn., the writer has designed and has now under construction somewhat more than half a mile of reinforced concrete pressure mains. There are two pressure pipes: One is a force main, extending from the site of the pumping station to that of the septic tanks, and has two diameters, 33 and 24 in.; the other main is to serve as a main influent or supply pipe for the sprinkling filters, and varies in diameter from 24 to 38 in.

The concrete section forming these pressure conduits varies gradually from the general section shown in Fig. 6, at one end, to that shown in Fig. 7, at the other end of the work. This particular aggregation of conduits was determined to be most economical for the work in hand owing to the peculiar governing conditions.

The plans and specifications did not contemplate the construction of the pressure mains in one continuous operation or as a monolith, but rather provided for definite stops in the work at intervals of 40 ft. and made provision at each of these stops for such expansion and contraction as might be caused by variations in the temperature, and also the contraction developing during the setting and hardening of the concrete. Each 40-ft. section has been given sufficient longitudinal steel to ensure the monolithic action of each individual section and the concentration of all expansion and contraction at the points provided to receive such action.

Mr. Taylor.

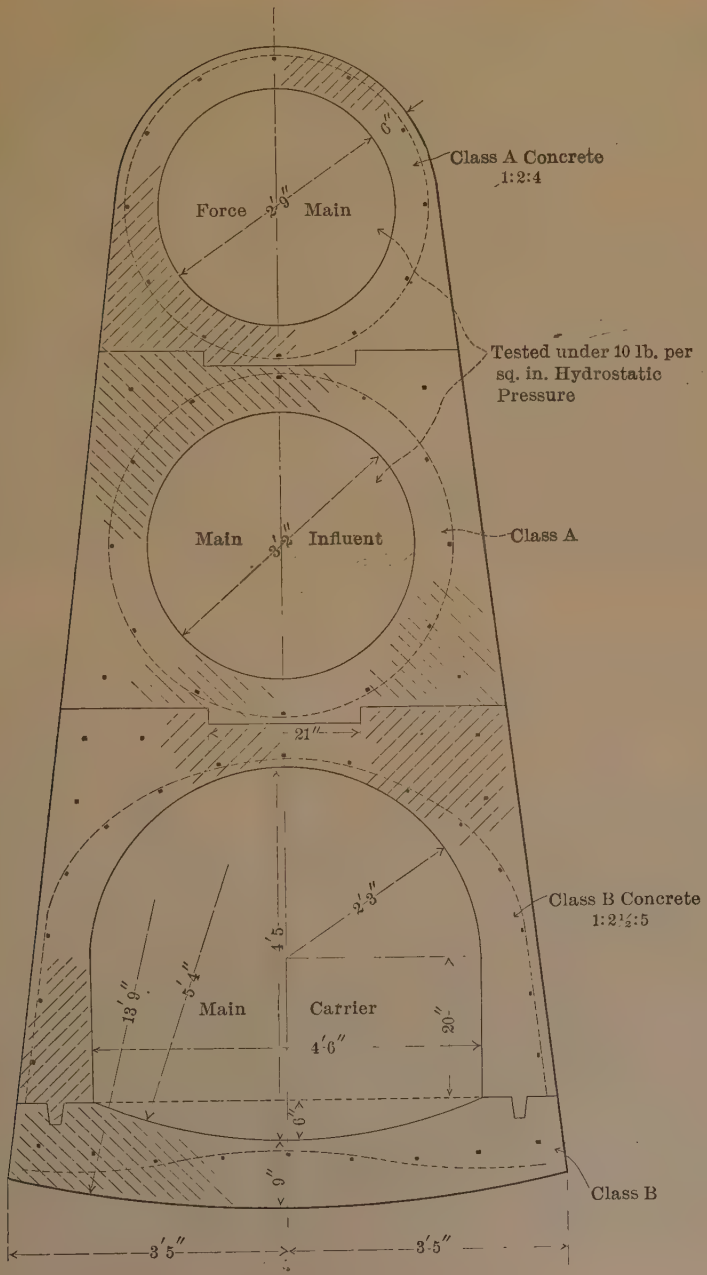


FIG. 6.

Mr. Taylor. Longitudinal reinforcement has been placed as shown in Figs. 6 and 7, and consists of twisted rods of high-carbon steel $\frac{5}{16}$ in. square. These rods have a tensile strength of 100 000 lb. per sq. in., and an elastic limit of one-half this amount. The rods have been received in 21-ft. lengths, bent cold at each end through an angle of 90° so as to have a projecting leg 1½ in. long, lapped 12 in., and wired together.

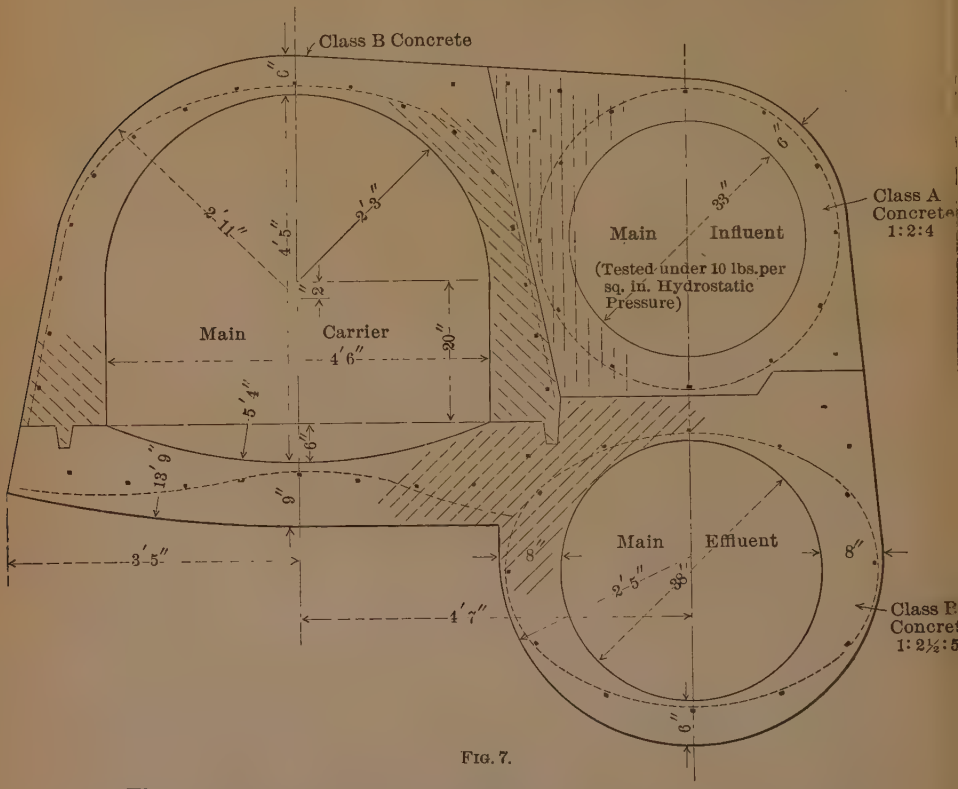


FIG. 7.

The transverse reinforcement consists of $\frac{3}{8}$-in. square, twisted rods of the same quality as the longitudinal rods. The spacing of these rods is such as to permit their receiving the maximum bursting stress with an accompanying total elongation of about 0.03 per cent. Thus the maximum working stress is low in the transverse steel and was designed in this way for the purpose of preventing the formation of longitudinal cracks on account of excessive elongation in the concrete.

The transverse rods have been bent through an angle of 90° at each end, to form a mechanical bond, and then bent to form the desired circle, around a series of iron pins driven into a wooden platform as shown in Fig. 1, Plate X. Previous to the placing of the

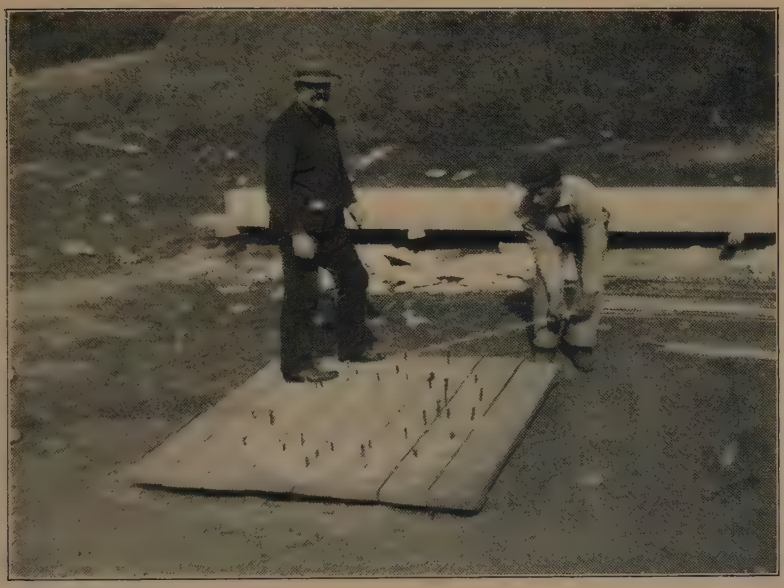


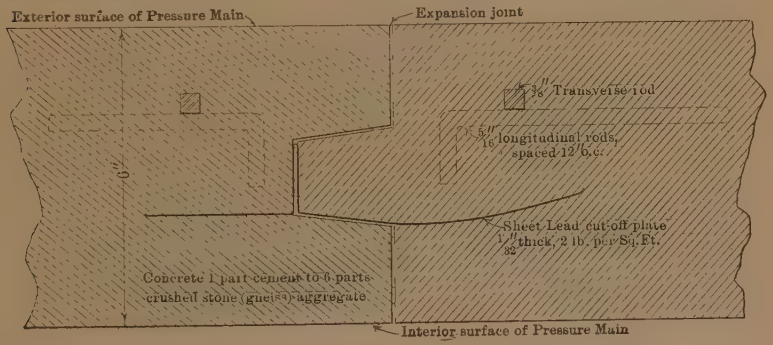
FIG. 1.—BENDING TRANSVERSE RODS.

FIG. 2.—BUILT-UP CAGE OF STEEL REINFORCEMENT FOR 42-IN. EFFLUENT MAIN.

steel for concreting, it was assembled, upon the bank, in the form of a cage, 40 ft. long and of the required diameter, by men doing this work alone. Mr. Taylor.

The forms for the pressure mains are of steel, in semi-circular sections 5 ft. long. Such forms are fastened together in 40-ft. lengths and used as units, being pulled ahead after the concrete of the previous section has become sufficiently set.

At the end of each 40-ft. section a mortise and tenon bonding joint of the ordinary type has been provided, and in addition to this a sheet-lead water cut-off plate has been inserted. The cut-off plates are 12 in. wide, $\frac{1}{32}$ in. thick and weigh 2 lb. per sq. ft. They have been bent into the shape shown in Fig. 8, the rear half of the flange being embedded in the concrete and the forward end allowed to project through the bulkhead form to be enclosed in the succeeding section of conduit.


Exterior surface of Pressure Main
Expansion joint
3/8" Transverse rod
5/16" longitudinal rods, spaced 12" c. c.
Sheet Lead cut-off plate 1/32" thick, 2 lb. per Sq. Ft.
Concrete 1 part cement to 6 parts crushed stone (gneiss) aggregate
Interior surface of Pressure Main

DETAIL OF EXPANSION JOINT IN PRESSURE MAINS.

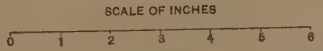


FIG. 8.

Subsequent contractions are thus concentrated at the expansion joints, and the lead water cut-off plates have readily adjusted themselves to the slight movements taking place there.

The concrete has been mixed in the proportion of 1 part Atlas cement to 6 parts of graded aggregate, the particles of which have ranged from 1½ in. in longest diameter to dust, and in such a manner as to produce a mixture of maximum density. The concrete has been mixed by machine to a very soft consistency, and has in all cases been placed so rapidly that the time required to complete a given 40-ft. section has not exceeded 2 hours. Upon the removal of the forms, the pressure pipes have been covered with burlap, and kept constantly wet by frequent sprinklings.

Mr. Taylor.

At inlets into the pressure mains, which have been constructed at frequent intervals, provision has been made for the lateral extension of water-tight distributing pipes by leaving a bonding joint and a lead expansion plate. Anchor bolts have also been built into the end of the inlet stubs and into temporary endings in the pressure mains to secure bulkheads and to facilitate the testing of the water-tightness of the conduit work in 500-ft. sections.

A view of the steel form, reinforcing metal, lead cut-off plate and pressure main bonding joint is shown in Fig. 2, Plate XI, and a detail of a lateral connection, 24 in. in diameter, is shown in Fig. 1, Plate XI.

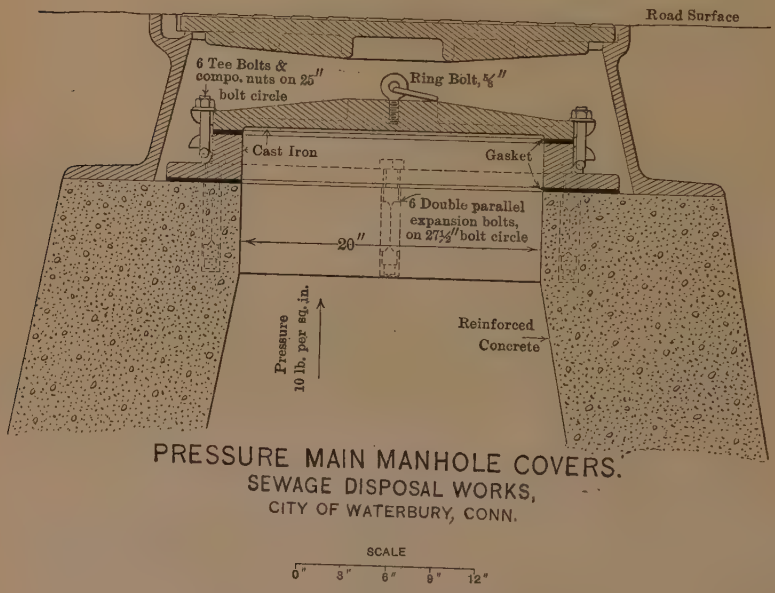


FIG. 9.

Manholes on the pressure mains have been constructed by a method similar to that used in the construction of the conduits. The method by which a manhole is capped with a convenient, accessible and water-tight cover is shown in Fig. 9.

The pressure conduits were designed for and have been tested under an internal water pressure of 10 lb. per sq. in., and under this pressure the present method of construction has been highly satisfactory. No longitudinal cracks have appeared, and transverse cracks have developed only at points where expansion joints have been provided. The water-tightness of these joints and the suitability of the lead water cut-off plates have been proven to be highly satisfactory.

FIG. 1.—AN INLET ON THE PRESSURE MAIN.

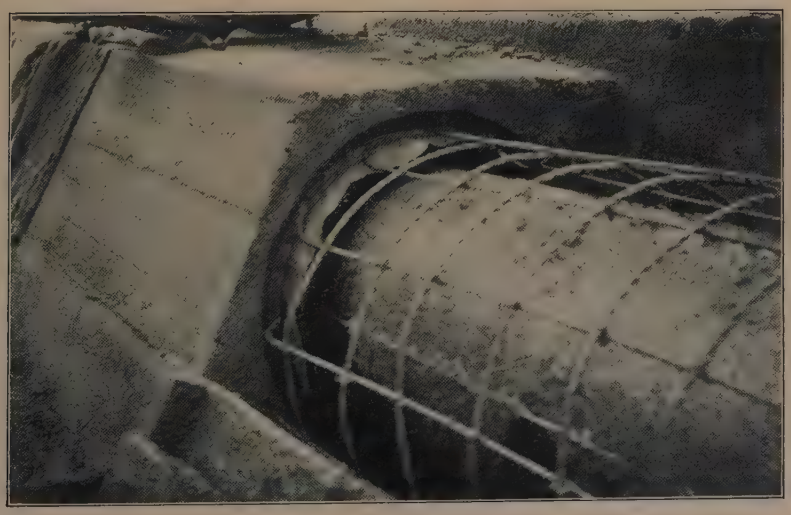


FIG. 2.—EXPANSION JOINT.

THOMAS H. WIGGIN, ASSOC. M. AM. SOC. C. E.—The pipe experiments of the Board of Water Supply have not reached such a stage as to justify final conclusions, but the speaker will describe briefly what has been done, so that those interested may perhaps have a chance to see a part of the work in progress. Mr. Wiggin.

There are several siphons along the proposed Catskill Aqueduct in which the head is 125 ft. or less, and for these it was apparently economical to adopt reinforced concrete structures, provided they could be made tight and safe. In order to determine this, some experiments on 5-ft. pipes were undertaken. The design of the experimental pipes was founded to a considerable extent on the work of J. H. Quinton,* M. Am. Soc. C. E., mentioned in Mr. Smith's paper. Plastering, however, was not accepted as desirable to accomplish watertightness. Ten experimental sections were made, each in two parts, so that joints, also, could be tested. Each part was 6 ft. long, and generally of different proportions from its mate, so that each of the ten test sections furnished a test of two kinds of pipe and one kind of joint. The pipes were built upon cradle-shaped supports in such a way that the whole could be observed, and leakage from each part and from the joint isolated. The proportions of the concrete were various, as were also the kinds of aggregate and the kinds of joints. Furthermore, two thicknesses were used, namely, 5 and 12 in., thought to be, respectively, the minimum practicable and the maximum which would be considered. In short, the attempt was made to get all the combinations possible in the pipes, while, at the same time, there were duplicates of the more important proportions in order to control the results.

Many of the pipes, as first turned out of the forms, were very imperfect. This seems to be inevitable in such work until the construction crew has had several months' experience. When the work was being planned, it seemed desirable to make the proportioning of the concrete as perfect as possible for maximum density. As the work progressed, however, it was realized that it was not so much the perfect proportioning of the concrete that was in question as the perfect filling of the forms, and an excess of sand, though proven by recently published experiments to detract from strength and density of test bars, seems to be a *sine qua non* in the thin, heavily-reinforced pipes. A concrete as rich as 1:6, which does not contain voids, will not leak under a head of 100 ft., but it is very difficult to avoid having porous spots, notably where the longitudinal steel holds up the coarse aggregate in the sides of the pipe. A concrete pipe is impervious where the concrete is present; but it is its absence that causes the trouble.

Comparatively simple repairs to these pipes reduced the leakage to a small fraction of the initial quantity. It was found that, by im-

* Water Supply and Irrigation Paper No. 143.

Mr. Wiggin. pregnating the porous places with grout from a small hand-syringe pump, using a grouting pad which fitted tightly against the concrete, a tremendous reduction in the leakage was effected, so that the results of some of the pipes 5 in. thick, made with 1:2:4 concrete, compared favorably with cast-iron pipes as usually found.

All the types of joint were unsatisfactory. They were designed on the principle that inevitably there would be frequent transverse cracks in lines of such pipe, and it was desirable to make some sort of joint which would permit, without leaking, opening due to contraction. With this end in view, there were made various bell-and-spigot joints, concreting the spigot part of the pipe first and the bell afterward, with the idea that any shrinkage would tend to make the contact better. To allow movement, three layers of water-proofing fabric, with asphalt inside, outside, and between them, were in some cases placed around the spigot before concreting the bell. In another case, only cold-water paint was used as a separator, and in still others were put complete rings of copper or lead made in the form of a **U** in such a way that expansion could take place without breaking them. In one pipe a caulked lead joint was used between two steel bands anchored carefully back to the longitudinal rods. The metal stops were successful, in so far as they were successfully embedded. Some leakage took place around them, due to mechanical imperfections. The speaker's conclusion with regard to joints is that it would be probably just as well to do the best work possible with a plain concrete joint, perhaps with some offset in it, and then to repair the contraction cracks by caulking in cold weather. There is room for discussion as to the desirability of attempting to reinforce across joints in changeable climates.

With the results of the above-described experiments in hand and also what was learned from the work described in Mr. Smith's paper, to which the engineers of the Board of Water Supply had access, plans were prepared for a larger experiment, which is now being conducted at Hunter's Brook, near Peekskill, N. Y. At this brook, the Catskill Aqueduct requires a siphon about ½ mile long which has a maximum head of about 125 ft. Two 11-ft. pipes are required to take the whole flow. A part of one of these lines is now being constructed. It is a pipe 8 in. thick at the top, sides, and bottom, and thicker on the haunches, which are sloped outward to enable the earth loads to be carried safely. The pipe is built on a 4-in. concrete floor, laid in the form of a wide trough, with sides sloping as steeply as can be screeded conveniently. The idea of this floor is to prevent, as far as possible, dirt, gravel, etc., from being washed down under the form and to enable any foreign matter that has accumulated to be flushed out so as not to reduce the thickness and purity of the bottom concrete. In the speaker's previous experience, the difficulty of cleaning out under the

form has been a very common source of trouble with such pipe, where there was no question of tightness, but merely a question of getting a good job. Mr. Wiggin.

The reinforcement is computed for a tension of 9 000 lb. per sq. in. Two kinds of pipes are being built; one with a single row of reinforcing rings and the other with a double row of rings. The rings of the single row are $1\frac{3}{8}$ in. square, and are about 4 in. apart. The rings of the double row are each 1 in. square, and are also spaced 4 in. apart in each row. The rings of the double row are set opposite each other, with the idea of obtaining a better chance to joggle the concrete than would exist if the rings were staggered. The mixture of concrete is $1:2:3\frac{1}{2}$—a little richer than in the Pinto and Cottonwood pipes. The idea was to have an excess of 1:2 mortar, the stone being considered merely as displacing material.

Four sections of this pipe have been cast, each 30 ft. long, and there will be from three to five more. To look casually at the form and steel ready for concreting a pipe, one would think that it would be a pretty difficult job to get the concrete in perfectly; but when one considers closely the surface of the concrete at any stage in the building of the pipe one can see that there is really plenty of room. The pipes are going to be good ones, practically considered, and will furnish a reliable test.

The main difference between the problem for the Catskill Aqueduct and that set forth in this paper is in the kind of water. If the tensile strength of the concrete is designedly passed, or even if it is passed only because of shrinkage and consequent compression of the steel, there will be longitudinal cracks in the pipes, even if invisible. If there is silt in the water, it will fill up these cracks. If there is no silt, it is a question whether a pipe of this sort can be made very tight or permanent without keeping the stresses very low and repairing imperfections carefully. The baneful effects of the shrinkage of the concrete, as referred to above and explained in the paper, are evident, as thereby the pressure at which the cracks begin is lowered, and the aggregate width of the cracks at a given pressure is greater than would be the case without shrinkage. The value of keeping such pipes damp during setting, so as to reduce this shrinkage, is therefore evident.

If some apparatus could be designed whereby the pipe would be expanded after it is built and grout forced into the cracks simultaneously, success would be assured.

The engineers of the Reclamation Service advise that the pipes be made thin, in order that the cracks shall be well distributed; in other words, so that the steel, rather than the concrete, shall be the predominant factor. The experiments of the Board of Water Supply showed that pipes 5 in. thick gave better results than those 12 in. thick.

Curves showing the variation of leakage with pressure in the 5-ft.

Mr. Wiggin. pipes of the Board of Water Supply indicate that the concrete did not crack until a tensile stress was reached varying in different cases from 160 to 340 lb. per sq. in. (computed by the thick, hollow-cylinder theory). A rough computation of the stress in the concrete of the Cottonwood siphon at its greatest depth indicates that the tension was about 140 lb. per sq. in., so that, eliminating shrinkage, it would not be expected that longitudinal cracks would be necessary in much of this siphon or in any part of the Pinto siphon.

Further statements in regard to the tests of the Board of Water Supply would better be postponed until a complete presentation of the experiments can be made, both on the 5-ft. and on the 11-ft. pipes. The speaker has no doubt that in due time the Society will be informed completely of whatever is learned.

The engineers of the Reclamation Service deserve great credit for the way the pipe was built and its cheapness under rather adverse conditions. It illustrates what can be done by ingenious engineers who are also practical. The alligator form obviates one of the principal difficulties with ordinary forms in obtaining perfectly filled pipe, namely, the difficulty of filling the invert. With the ordinary form, in which the inside form is wholly erected before concreting, thus making liquid concrete necessary, voids commonly occur in the invert, unless the foreman is careful and observing, and has had long training in the same kind of work. On the other hand, the concrete in the invert is visible ahead of the alligator, and, in fact, all the concrete is directly in sight when placed. The speed attained, with cement of the right speed of setting for that particular work, is phenomenal, and it should be possible, generally, to obtain reliable cement suitable for rapid progress.

The author has introduced one word which the speaker thinks is worthy of adoption for specifications by engineers generally. That is the word, "churning." One still sees and uses the word, "ramming," in specifications, though it is a misnomer with concrete mixed wet, as is usual nowadays.

The wetting of the concrete during setting is of prime importance, as indicated by the author. The pipes should be kept wet in order that the shrinkage may be reduced to a minimum.

It was evident that the cold-weather concrete was much freer from transverse cracks than the warm-weather concrete, and that the longitudinal reinforcement did not prevent such cracks. This seems to be inevitable, and points to the desirability of laying such pipe in reasonably cold weather, where possible.

The author takes exception to Mr. Quinton's statement in regard to reliance on internal plastering. He still washes the interior of his pipes with cement. The latter practice, also, in the speaker's opinion, is questionable, because he has often seen such a wash peel off in spots,

and fears that it may lead to fancied security which will not be permanent. Experience will show whether the coating of cement is likely to come off under conditions of continual wetness as it does in the atmosphere. Mr. Wiggin.

In conclusion, it can be said that experience with the pipes described by the author proves that, with low heads and silt-bearing water, reinforced concrete pipes are entirely practicable and are an economic type of structure. For high heads, and for water which is not silt-bearing, the question still remains open. Even if reinforced concrete can be made tight for high heads, its economy is questionable. If designed for a tension of 10 000 lb. per sq. in., reinforced pipes require more steel than sheet-steel pipes which are not too thick for the tension, and it costs fully as much to put the steel into the reinforced pipe as into a steel pipe. As the concrete is necessary in addition, the economy for high heads is doubtful, even taking into account the larger renewal factor for steel pipes. For low heads, the quantity of steel in the reinforced concrete pipe can still be only as needed for the tension, whereas the quantity in the steel pipes cannot be small on account of practical limits. Taking the case in point, of a pipe 5 ft. 3 in. in diameter: A steel pipe $\frac{1}{4}$ in. thick is good for a head of about 160 ft., allowing $1\frac{1}{16}$ in. for rust, seven-tenths for joint efficiency, and a tension of 16 000 lb. per sq. in. on the net metal. Plate $\frac{1}{4}$ in. thick is thinner than would commonly be used, and the head is about twice as great as the head upon any part of the pipes described by the author. Hence, the economy of a reinforced concrete pipe in this particular case seems evident, whereas, with heads high enough so that a steel pipe of practical thickness would be worked to its capacity, the economy would begin to disappear.

In reference to the prospect of the reinforcement being "corroded" where a longitudinal crack opens up, the experiments of the Board of Water Supply show that there is some iron coming from these leaks, sufficient, in fact, to cause apprehension. The leakage must be stopped in order to make the reinforcement safe. Experience alone will tell what the life of such pipes will be.

Chester Wason Smith, Assoc. M. Am. Soc. C. E. (by letter).— Mr. Smith.
Regarding the reasons advanced by Mr. Teichman against using a sliding mould, the writer agrees with him that the concrete is likely to be disturbed. With the form used in building these pipes, the moving lower part disturbed the upper stationary plates more or less, that is, small obstructions (such as a pebble or a spatter of concrete) on the rail jarred the upper plates. Also (which Mr. Teichman probably intended to mention), it was difficult to make an accurate adjustment of the support between a lower stationary plate and its corresponding upper plate, so that, after the alligator had rolled from under it, it would remain in exactly the same position it had formerly occupied.

Mr. Smith. The moving part, at times, may mar the appearance of a concrete surface which is in contact with it, or it may have some slight inequality which may catch up a pebble in the concrete and roll it until the form rolls off. The writer cannot agree with Mr. Teichman that it has any tendency to affect injuriously the compactness and density of the concrete. The moving form acts on the concrete as does a trowel upon mortar, in fact, it renders that portion of the concrete actually more dense than when it is deposited against stationary plates.

Regarding the immunity of the Cottonwood lines from transverse cracks, it should probably be attributed almost entirely to the favorable temperature at which they were built, namely, 35 to 55 degrees.

Several points are suggested by Mr. Jonson's discussion: First, for a pipe intended to carry water which is free from sediment, it might be worth considering whether or not sediment could be introduced when the pipe is first filled; that is, fill the pipe with water made quite muddy, and let it stand for a time, possibly agitating the mud.

It should not be difficult to eliminate the longitudinal cracks; the transverse ones, occurring at joints between work of different days, need not intersect the circumferential reinforcement, and it is not of vital importance if they expose the longitudinal reinforcement, except in the case of a questionable foundation and the consequent possibility that the pipe might have to support itself in places as a beam.

Mr. Jonson suggests various refinements and additions which might have been applied to the leakage tests; but, it seems to the writer, these would have been valueless without a lot of other refinements not warranted under the circumstances and rendered impracticable by the conditions.

Regarding the leakage per unit of length, and the average head: sufficient information is given in the paper to enable one to obtain those figures.

Mr. Lesley suggests that a reason should have been given for building the pipe in place, rather than in sections afterward to be placed in the trench and connected. A sufficient reason was that the cost was much less. Suppose a pipe of the same size, with a shell 3 in. thick: such a pipe, with reinforcement, would weigh about 700 lb. per lin. ft. Assume a practicable length to be 5 ft., or $1\frac{3}{4}$ tons: it would be expensive to transport it from the yard and put it in the trench. A joint would have to be made for each 5 ft., that is, there would be eight times as many joints as there were by building in place 40 lin. ft. per day. These joints might be made tight, but it would be expensive, especially if the longitudinal reinforcement was joined, which probably should be done. It would require considerable attention to bed such sections properly, and they would need careful tamping

under them after being laid, to insure that none settled under the load and possible accompanying leaks. It might require a concrete bed to support them properly. Mr. Smith.

Homer A. Reid, Assoc. M. Am. Soc. C. E.,* gives the cost of 6½-ft. unlined pipe, in Paris, as about $12 per lin. ft., but does not give the items. If it cost that much in Paris, probably with trained workmen, the reader may imagine what it would probably cost in Arizona, with the inefficient and high-priced labor which must be employed there.

Allowing a saving of one-half in the cement used, the cost would have been two or three times as great as it actually was.

Mr. Wiggin gives an interesting preliminary report on the investigations of the New York Board of Water Supply. The results of these experiments should be very valuable, and the profession will welcome the detailed account which Mr. Wiggin promises will be presented at the proper time.

If the reinforced concrete type of construction proves successful, the resulting increase in flexibility of alignment of a conduit may be utilized in dodging artificial obstacles as well as overcoming natural ones; and this should open up many possibilities in laying out lines through expensive rights of way.

TABLE 3.—SOME LATER OBSERVATIONS ON THE QUANTITY OF LEAKAGE.

Pipe Line.	Date.	Time, in hours.	Gallons.	Gallons per hour.	Average head, in feet.
Pinto Pipe, North Line..	March 29–30, 1907.	24	1 464	61	30.5 to 29.3
Pinto Pipe, South Line..	March 6–27, 1907.	1	792	792	30.5 to 29.9
		1	700	700	29.9 to 29.4
		1	609	609	29.4 to 28.9
		15	7 035	469	28.9 to 23.9
		1	517	517	23.9 to 23.5
		2	1 126	563	23.5 to 22.7
		3	1 461	187	22.7 to 21.7
Both Cottonwood Lines.	April 1–2, 1907....	24	0	0	74.0

* "Concrete and Reinforced Concrete Construction," page 646.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1066.

REINFORCED CONCRETE TOWERS.*

By D. W. Krellwitz, Jun. Am. Soc. C. E.

With Discussion by Messrs. R. D. Coombs, and D. W. Krellwitz.

Electrical energy is now being delivered by transmission lines to cities many miles distant from its source. The wires carrying the current are supported on towers which must conform to the requirements of navigation when rivers and canals are crossed, that is:

"There must be ample overhead clearance from the water to the nearest point of the transmission wires, so that vessels with high masts can pass under the wires." (Fig. 1.)

This paper describes the reinforced concrete towers, built for The Lincoln Light and Power Company, on each side of the old Welland Canal, in the Province of Ontario, to transmit current from the transformer building to St. Catharines, Ontario, Canada.

These towers are 150 ft. high, 142 ft. being above ground. They are 11 in. square at the top and 31 in. square at the bottom, with chamfered corners, and are provided with foot steps and rungs. Each tower carries sixteen No. 1 bare copper wires on glass insulators. The cross-arms are $3\frac{1}{4}$ by 4 in., and are 10 ft. long, and beneath them there is a working platform about 10 ft. long and 5 ft. wide. The span over the canal is 76 ft., but adjacent to this

*Presented at the meeting of October 2d, 1907.

there is a span of about 300 ft. On one of the towers the wires run vertically, being fastened to two frames, 42 and 82 ft., respectively, from the top. At the top of this tower there is a heavy pull parallel to the line, which at this point makes a right-angled turn.

The collapse of a tower on a transmission line, caused by a storm, would be a very serious matter, and would be likely to cause lawsuits, the loss of light and power customers, and, perhaps, loss of life, on account of the dangerous currents.

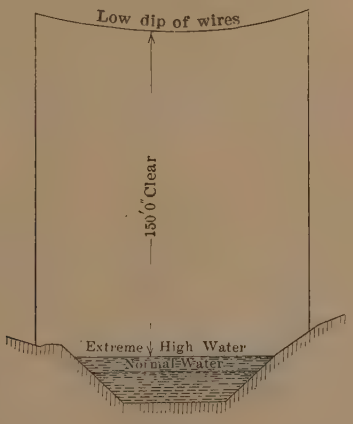


FIG. 1.

These towers have been designed to withstand wind pressures of 30 lb. per sq. ft. on flat surfaces, and 15 lb. per sq. ft. on cylindrical surfaces. A liberal assumption was made by calculating the exposed surface of the wire as its diameter plus a coating of ice $\frac{3}{8}$ in. thick multiplied by the span. As the greatest wind pressures in that part of the country have never exceeded an average of 30 lb. per sq. ft. on large areas, it is reasonable to believe that these towers are absolutely safe.

The following conditions were observed in designing the towers:

Case I.—Tower, without wires, and not guyed; to withstand a pull of 2 000 lb. at the top;

Case II.—Guyed tower; to withstand full wind pressure plus pulls transverse to the line, due to the wires;

Case III.—In case of the breakage of all the line wires—for 300 ft. span—the total pull to be taken on the guyed tower plus full wind pressure on the tower.

The calculations are based on the following assumptions:

First.—The section plane before bending remains plane after bending; that is, the stress of any fiber is proportional to its distance from the neutral axis (Fig. 2).

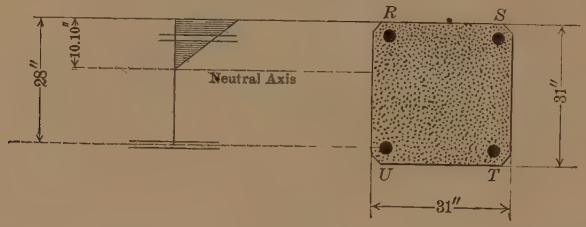


FIG. 2.

Second.—The applied forces are perpendicular to the plane of the neutral axis.

Third.—The tensile strength of the concrete is neglected.

Fourth.—There are no initial strains.

$\frac{\text{Modulus of elasticity of steel}}{\text{Modulus of elasticity of concrete}}$ = ...	15
Ratio of cross-section of steel in tension to cross-section of beam above this steel...	0.01129
Ratio of cross-section of steel in compression to cross-section of beam above the steel in tension	0.01129
Compressive stress in the extreme fiber of concrete.....	600 lb. per sq in.
Tensile stress in steel.....	16 000 lb. " " "
Compressive stress in steel.....	6 350 lb. " " "
Ratio of depth of steel in compression to depth of steel in tension.....	0.10714
Depth of steel in tension.....	28 in.

Case I.—The bending moment is $2\,000 \times 142.0 = 284\,000$ ft-lb. The distance from the compressive surface to the neutral axis, when the maximum allowable bending moment is applied, is:

$$\left[\sqrt{2 \times 15 \times (0.01129 + 0.01129 \times 0.10714) + 15^2 \times (0.01129 + 0.01129)^2} - 15 \times (0.01129 + 0.01129)\right] \times 28 = 10.108 \text{ in.}$$

The ratio of depth of neutral axis to depth of steel in tension is..... 0.3610

The moment of resistance is:

Taking moments about the center of compression in concrete,

$$\frac{31 \times 28^2 \times \left[16\,000 \times 0.01129 \times \left(1 - \frac{0.361}{3}\right) + 6\,350 \times 0.01129 \times \left(\frac{0.361}{3} - 0.10714\right)\right]}{12} = 323\,700 \text{ ft-lb.}$$

Taking moments about the center of compression stress in the steel,

$$\frac{31 \times 28^2 \times \left[16\,000 \times 0.01129 \times (1 - 0.10714) - \frac{600 \times 0.361}{2} \times \left(\frac{0.361}{3} - 0.10714\right)\right]}{12} = 323\,700 \text{ ft-lb.}$$

The compressive stress in the extreme fiber of the concrete, due to bending, is:

$$\frac{284\,000 \times 12}{31 \times 28^2 \times \left[\frac{0.361}{2} \times \left(1 - \frac{0.361}{3}\right) + \frac{15 \times 0.01129 \times (0.361 - 0.10714) \times (1 - 0.10714)}{0.361}\right]} = 529 \text{ lb. per sq. in.}$$

The compressive stress in the steel, due to bending, is:

$$\frac{284\,000 \times 12}{31 \times 28^2 \times \left[0.01129 \times \left(\frac{1 - 0.361}{0.361 - 0.10714}\right) \times \left(1 - \frac{0.361}{3}\right) + 0.01129 \times \left(\frac{0.361}{3} - 0.10714\right)\right]} = 5\,578 \text{ lb. per sq. in.}$$

The direct compressive stress in the concrete, caused by the dead load of the tower above the point of maximum stress, due to bending, is:

$$\frac{63\,300}{(19.6 \times 15) + (31^2 - 19.6)} = 51 \text{ lb. per sq. in.;}$$

19.6 sq. in. = area of four rods;

63 300 lb. = dead load.

The direct compressive stress in the steel, caused by the dead load of the tower above the point of maximum stress, due to bending, is:

$$\frac{63\,300}{19.6 + \frac{31^2 - 19.6}{15}} = 769 \text{ lb. per sq. in.}$$

The combined compressive stress in the extreme fiber of the concrete is:

$$529 + 51 = 580 \text{ lb. per sq. in.}$$

The combined compressive stress in the steel is:

$$5\,578 + 769 = 6\,347 \text{ lb. per sq. in.}$$

Case II.—

Diameter of wire	0.3 in.
Thickness of ice coating	0.75 "
Total	1.05 in. = 0.087 ft.

Wind pressure per linear foot of wire $= 0.087 \times 15 = 1.3$ lb.

Weight of wire per foot	0.27 lb.
Weight of ice load per foot	0.324 "
Total	0.594 lb.

The resultant force, normal to the line, is:

$$\sqrt{0.594^2 + 1.3^2} = 1.43 \text{ lb. per ft. of wire;}$$

and the total resultant force is:

$$\left(\frac{300 \text{ ft. span}}{2} + \frac{76 \text{ ft. span}}{2} \times 1.43\right) \times 16 \text{ wires} = 4\,300 \text{ lb.}$$

Vertical force on tower, due to resultant force	5 000 lb.
Weight of cross-arms and platform	600 "
Dead load of tower above the point of maximum fiber stress due to bending	63 300 "
Total	68 900 lb.

MOULD FOR REINFORCED CONCRETE TOWER.

Area of four steel rods = 19.6 sq. in.

The fiber stress in the concrete, due to direct compression, is:

$$\frac{68\,900}{(19.6 \times 15) + (31^2 - 19.6)} = 56 \text{ lb. per sq. in.}$$

In calculating the fiber stress due to bending, caused by wind pressure, the towers are considered as held at the top with steel guys and fixed at the base (Fig. 3).

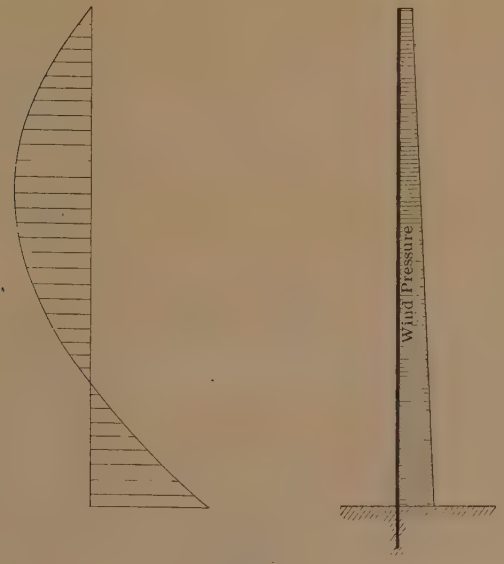


FIG. 3.

The wind pressure is:

247 sq. ft. × 30 lb. = 7 410 lb.

and the maximum moment due to wind is 118 200 ft-lb.

The fiber stress due to direct compression is	56 lb. per sq. in.
The fiber stress compression due to bending is	222 " " " "
Total compression in the concrete.	278 lb. per sq. in.

Case III.—

The area of one wire is 0.07 sq. in., and, in case all the line wires break, the pull is:

$$0.07 \times 16 \times 60\,000 = 67\,200 \text{ lb.}$$

The vertical force on the tower is.....	50 000 lb.
The weight of the cross-arms and platform is	600 "
The dead load of the tower, above the point of maximum fiber stress, due to bending, is	63 300 "
Total.....	113 900 lb.

The fiber stress in the concrete due to direct compression is:

$$\frac{113\,900}{(19.6 \times 15) + (31^2 - 19.6)} = 92 \text{ lb. per sq. in.}$$

The maximum moment due to wind is the same as for Case II, or 118 200 ft-lb.

The fiber stress due to bending is...	222 lb. per sq in.
The fiber stress in the concrete due to direct compression is.....	92 " " " "
The total compression in the concrete is	314 lb. per sq. in.

The unsupported length of tower is fifty-two times the maximum width and seventy-seven times the average width. It would be of interest to know what "verified formula" would give the permissible strains per square inch for long reinforced concrete compression members.

A square cross-section, with four corner rods (*R, S, T,* and *U*), was chosen for the towers, as such an arrangement was more economical in steel than any other, each rod doing double duty. For instance, if the tower were strained about one axis, the rods, *R* and *S,* being in tension, if strained about another axis at right angles to the first, the rods, *S* and *T,* being in tension, the rod, *S,* would be doing double duty.

FIG. 1.—SHEARS FOR ERECTING TOWERS

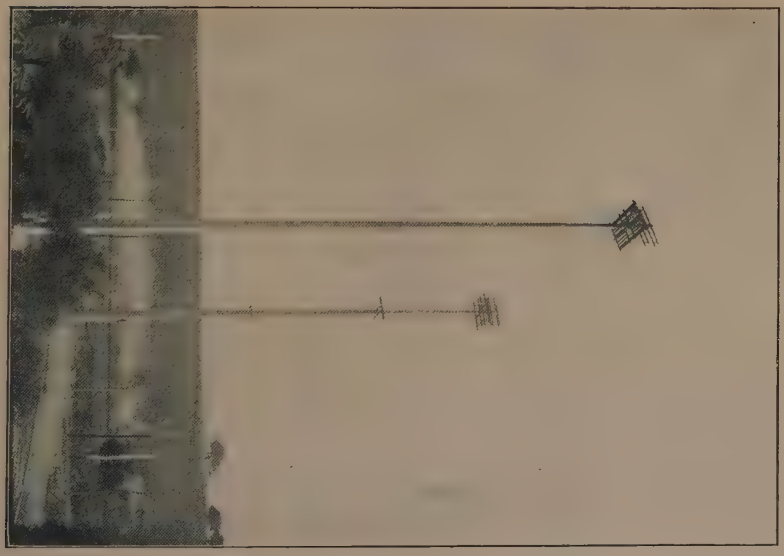


FIG. 2.—REINFORCED CONCRETE TOWERS.

The tower at the right-angled turn of the line stood a remarkable test while changing the guys. On August 2d, 1906, it withstood the pull (tension in wires) parallel to the line without any guy to resist this pull. The maximum deflection at the top was about 2 ft. Not having the necessary apparatus at this time, the pull could not be ascertained. There were no visible cracks, nor was there any movement at the base. On another transmission line, however, a 59-ft. tower showed movement at the base, because the latter was not large enough.

The base of each tower was constructed with an opening on one side so that the foot of the tower could not slide horizontally during erection. The finished base is a cube of concrete 10 ft. on each side, and having 8 ft. of the tower within it.

The moulds for the towers were inclined from the base upward, as shown by Plate XII, because the saving in erection more than offset the extra cost of excavating. While setting up the bottom sections on blocking, great care was taken to avoid unequal settlement. The side sections were set plumb, and braced properly to withstand the lateral pressures. The moulds were wet (except in freezing weather) before placing the concrete. Cores were set to wooden templets near the top for the holes required for bolting the cross-arms. Holes, 2 in. in diameter, were bored in the top side sections so that the bent ends of the foot steps would pass through when the moulds were removed.

The concrete for the towers was composed of 1 part of Portland cement and 5 parts of the very best gravel, with sand, of which 36% was "fine," having passed through a sieve of 0.2-in. mesh, and 64% was "coarse," being that which was retained in the sieve. The gravel, sand, and cement were first thrown together and turned over twice; water was supplied in pails, and the mixture was turned a sufficient number of times to produce a loose concrete of uniform color and consistency. Great care was taken in mixing the concrete and also in placing the steel and concrete.

At intervals of 20 in., foot steps were embedded in the upper part, and ladder rungs in the lower part, of each tower.

Two wooden shear legs, Fig. 1, Plate XIII, with necessary steel back and side guys and steel hoisting tackle, were used in erecting the towers. Careful study was made during erection

as to the proper positions of the hitches, in order to avoid any excessive deflections which might be caused by concentrated pulls during erection.

The last tower was moulded on April 17th, 1906, and the sides of the mould were removed two days later. Its erection was commenced on May 25th, 1906, or 38 days after moulding; smaller towers, however, have been erected successfully 14 days after moulding.

It is believed that these towers are the highest monoliths in existence at the present time, and it may be interesting to state that during their erection no men were injured in any way.

DISCUSSION.

R. D. Coombs, Assoc. M. Am. Soc. C. E. (by letter).—The author states that "the greatest wind pressures in that part of the country have never exceeded an average of 30 lb. per sq. ft. on large areas." While the writer is very ready to admit the probability of this statement, he would inquire whether the author has any data on recorded velocities in the locality mentioned, as the wind pressure used would correspond approximately to a velocity of 102 miles per hour. Mr. Coombs.

It is not perhaps a part of the immediate subject of the paper, but it would be of interest to know the working stresses and factors of safety allowed in the span wires, insulators and other connections.

The author's assumption that all wires may break at once, at their ultimate tension, seems to the writer to be entirely unwarranted. Such an accident could only occur from:

First.—Some object striking the wires: This is very unlikely at the altitude given, and is practically impossible on account of the different planes in which the wires lie.

Second.—By the wires breaking under ice and wind loads: If the factor assumed for the wires is low enough, this is possible, though the breaking of one wire would in all likelihood jar the ice from the adjacent wires.

It seems probable that, theoretically at least, the factor of safety assumed for the tower is greater than that of the individual connections of the wires to the glass insulators. In this case the insulators or cross-arms would be torn off before the tower received the full load assumed in Case III.

D. W. Krellwitz, Jun. Am. Soc. C. E. (by letter).—Mr. Coombs states that: Mr. Krellwitz.

"It seems probable that, theoretically at least, the factor of safety assumed for the tower is greater than that of the individual connections of the wires to the glass insulators."

This is true, and the writer does not know any good reason for doing otherwise.

Case III makes a very liberal assumption, and, although it may seem unwarranted, it certainly exhausts all possibilities; and, as this liberal assumption does not increase the cost of the towers, the owner might just as well have it.

In reference to the inquiry relating to wind velocities, it may be stated that the maximum recorded wind velocity for the locality mentioned, according to the United States Weather Bureau,* is 80 miles per hour.

* "Storms of the Great Lakes," by E. B. Garriott; prepared under the direction of Willis L. Moore, Chief, U. S. Weather Bureau.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1067.

WATER PURIFICATION AT ST. LOUIS, MO.*

By Edward E. Wall, M. Am. Soc. C. E.

With Discussion by Messrs. Philip Burgess, Edward Prince, George A. Soper, G. C. Whipple, L. L. Tribus, L. J. Le Conte, and Edward E. Wall.

Previous to 1904, St. Louis was famous for the muddy water supplied to its inhabitants and visitors. The inhabitants, never having known anything else, had ceased to pay attention to the appearance of the water, accepting it as a matter of course, just as they endured the wind and the weather, and were inclined to be amused at "the stranger within their gates," who protested against drinking and bathing in the coffee-colored fluid. Visitors to St. Louis in the summer and autumn of 1904, who had previously been in the city, were amazed to find clear, sparkling water in general use even for sprinkling and street washing. The history of events leading up to this sudden and remarkable change in the water supply of St. Louis, and a description of the purification process as finally developed and used, cannot fail to be of interest to engineers, especially to those engaged on waterworks and in the investigation of water supplies.

In May, 1903, Mr. Ben C. Adkins was made Water Commissioner of St. Louis, and the writer was appointed by him as Principal Assistant Engineer, the position of Assistant Water Commissioner being created later.

*Presented at the meeting of November 6th, 1907.

The most important question confronting the Department at that time was to devise some method of improving the appearance of the water, which should be of such design that it could be put into operation previous to the opening of the World's Fair, that is, in less than one year.

A brief description of the water-works will be necessary in order that the conditions existing at that time, and the changes that were afterward made, may be understood. The water supply of St. Louis is taken from the Mississippi River at the Chain of Rocks, about ten miles above the Eads Bridge, which spans the river approximately at the center of the city. The intake tower stands near the middle of the river, resting on bed-rock and built over a 7-ft. shaft leading down to a tunnel some 50 ft. below the river bed. This tunnel is also 7 ft. in diameter and terminates at the river bank at the base of the uptake shaft, which connects the river and the shore tunnels. The shore tunnel, also 7 ft. in diameter, is about 30 ft. nearer the surface than the river tunnel, and ends at the wet well from which the low-service pumps take their supply. (Plate XIV.)

The low-service pumps deliver the water to a well at the head of a 9 by 11-ft. horseshoe-shaped conduit, which runs along the ends of a series of six settling basins, each 400 by 670 ft., with connections and gates for filling each basin separately. At the opposite ends of these basins is another 9 by 11-ft. conduit with connections to each basin for drawing off the water. This drawing conduit carries the water to the high-service stations several miles below. (Fig. 1.)

Previous to 1903, the operation of the settling basins consisted in a separate filling and drawing of each basin, allowing as long a time as possible for settlement before drawing off the water for delivery to the high-service pumps. This time usually varied from 12 to 24 hours, during which the heaviest and coarsest portion of the suspended matter fell to the bottom, but so much finely-divided matter remained in suspension that the casual observer could scarcely notice any difference between the river water and that delivered to the consumer.

The river water carries in suspension amounts varying from 20 to 6 000 parts per million, according to the stage of the river, the solids in solution varying from 140 to 400 parts per million, the bacteria ranging from 2 000 to 250 000 per cu. cm.

The method of simple sedimentation reduced the suspended solids

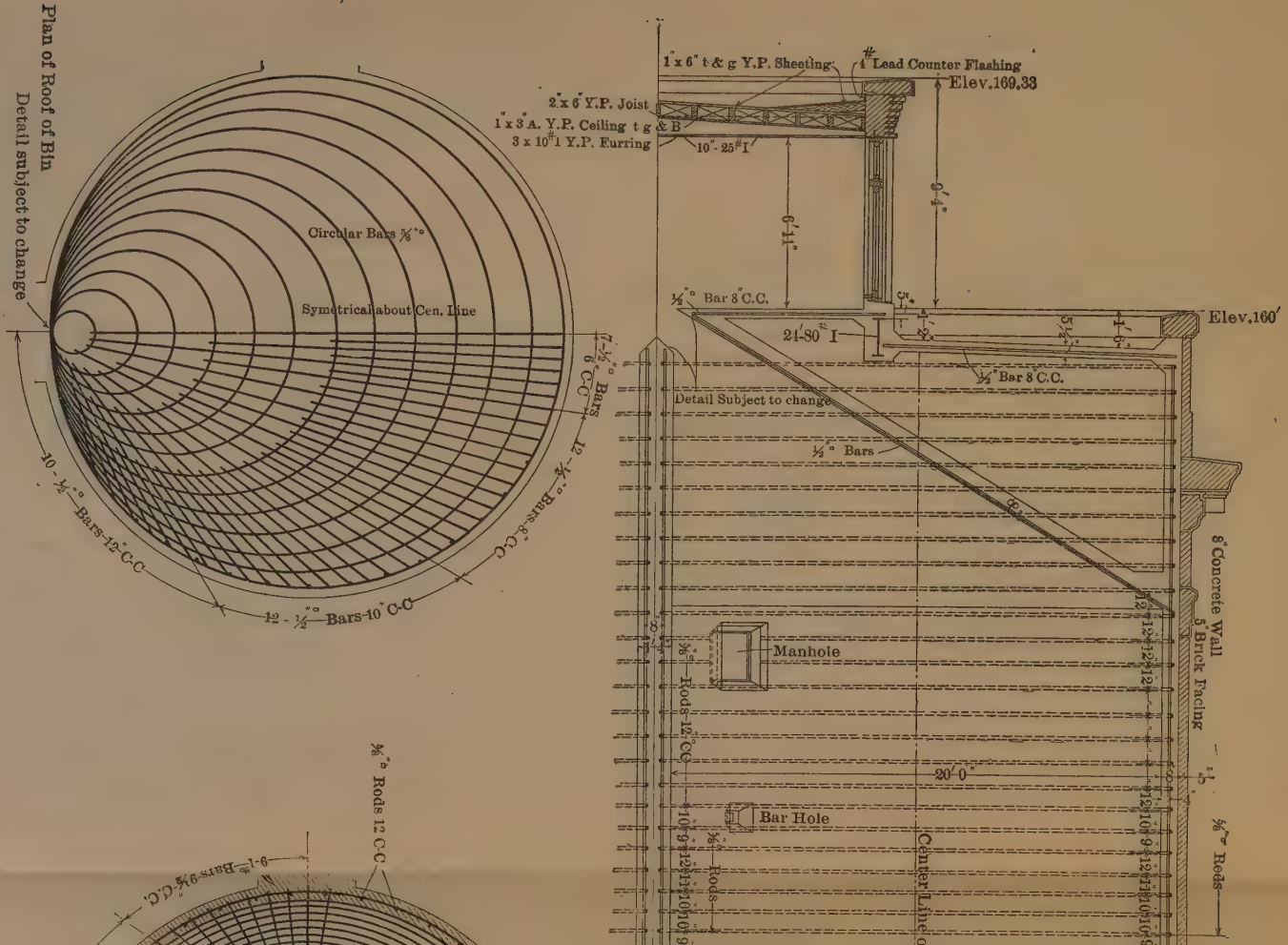
from 10 to 80%, the number of bacteria being reduced probably in the same ratio, but was eminently unsatisfactory because of the low percentage of improvement and the scarcely perceptible change in the appearance of the water.

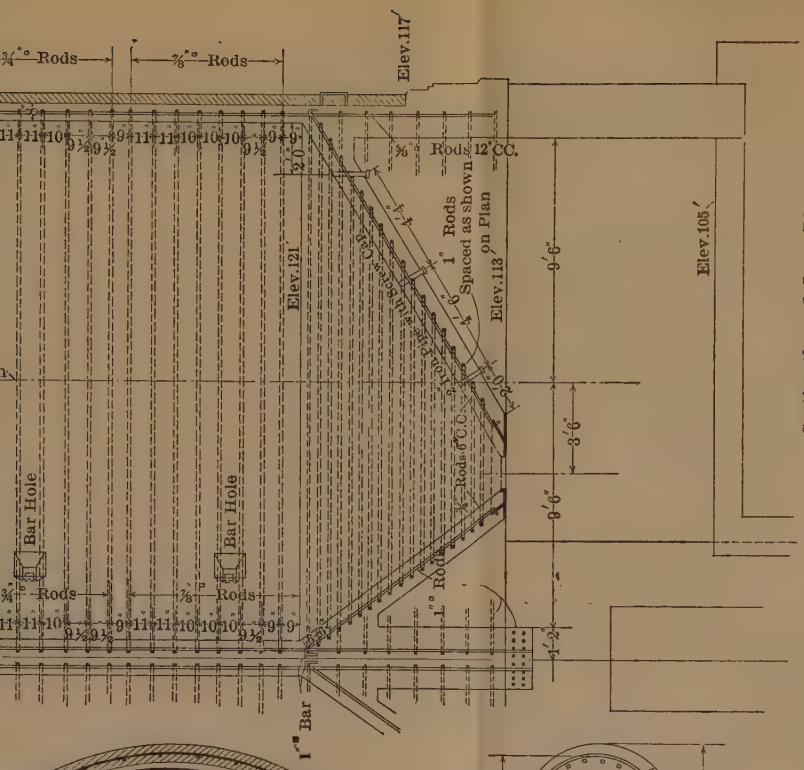
The problem of devising a better system of clarifying and purifying the water supply of St. Louis had been studied and discussed by the best hydraulic engineers in the country. Filtration plants at home and abroad had been examined and studied, and various projects had been offered as solutions of the problem. For 40 years the question had occupied a prominent place in all propositions connected with the improvement and extension of the water-works. All proposed schemes for clarifying and purifying the water supply, which were worthy of serious consideration, involved the expenditure of millions, and necessitated the building of works which would take years to complete.

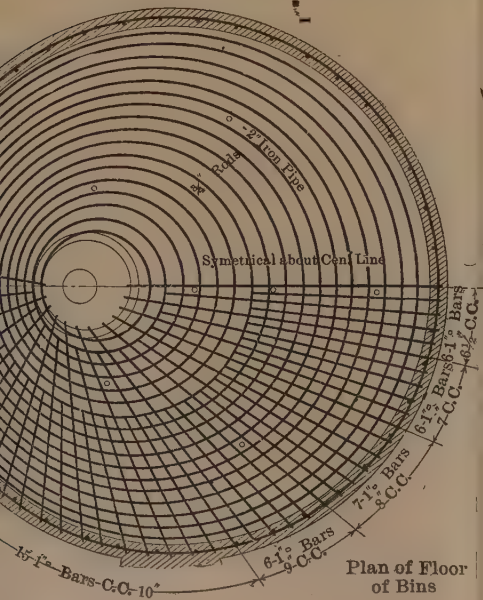
Many years ago it was proposed to abandon the Mississippi River as a source of supply and to go some 90 miles away to the Meramec Springs. Another proposition was to move the intake to the Missouri River above St. Charles, Mo. These schemes contemplated furnishing the water to the city by gravity flow, the Meramec project involving the construction of an immense impounding reservoir with miles of masonry conduit, steel pipe and tunnels, while the Missouri River plan required the building of a new low-service pumping station to deliver water to a set of new basins to be constructed on high ground a few miles outside the city.

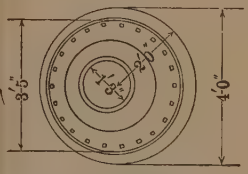
It was not until about 1901 that the Meramec project assumed a shape definite enough to attract sufficient attention to warrant a detailed investigation of its merits. It developed that this plan called for an expenditure of more than $30 000 000 and the abandonment of the existing pumping stations. As an alternative it was proposed to build a mechanical filtration plant to cost approximately $2 000 000, with an estimated cost of operation of $7.43 per 1 000 000 gal., amounting to an annual charge of about $200 000. In either case, aside from the expense, it would be a matter of years before the city could expect material improvement in the quality of its water supply.

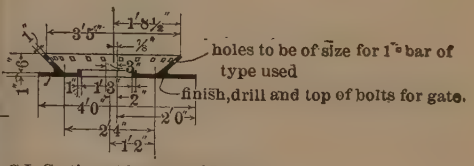
Under the conditions thus briefly described, the present Water Commissioner assumed office in May, 1903. Work was started at once upon a plan designed for a system of flowing sedimentation, the settlement to be hastened by the use of some coagulant, presumably sulphate of


Plan of Roof of Bin
Detail subject to change
Circular Bars ⅜"ø
Symetrical about Cen. line
7 - ½"ø Bars 6" C.C.
12 - ½"ø Bars 8" C.C.
12 - ½"ø Bars 10" C.C.
10 - ½"ø Bars 12" C.C.
⅜" ø Rods 12 C.C.
9-1⅝" Bars 9½" C.C.
1" x 6" t & g Y.P. Sheeting
4" Lead Counter Flashing
Elev. 169,33
2" x 6" Y.P. Joist
1" x 3" A. Y.P. Ceiling t g & B
3 x 10" 1 Y.P. Furring
10" - 25# I
6'-11"
6'-4"
½" ø Bar 8" C.C.
24'-80# I
Elev. 160'
½" ø Bar 8" C.C.
Detail Subject to change
½" ø Bars
Manhole
⅜" ø Rods 12" C.C.
20'-0"
Bar Hole
⅜" ø Rods
Center Line
8' Concrete Wall
5" Brick Facing
¾" ø Rods









C.I. Casting at bottom of Bins

COAGULANT HOUSE
SECTIONS OF LIME BINS & DETAILS
CHAIN OF ROCKS

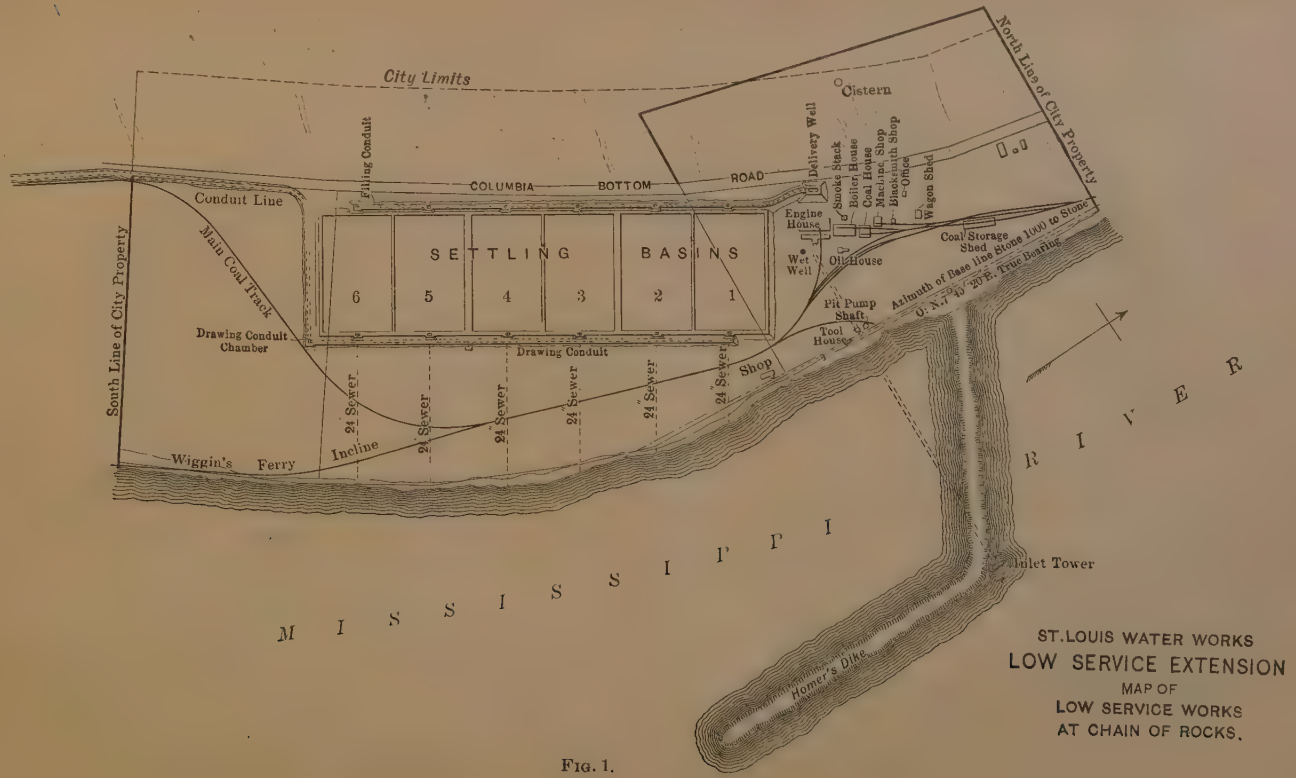

City Limits
North Line of City Property
Cistern
Filling Conduit
COLUMBIA BOTTOM ROAD
Delivery Well
Smoke Stack
Boiler House
Coal House
Machine Shop
Blacksmith Shop
Office
Wagon Shed
Engine House
Conduit Line
Main Coal Track
SETTLING BASINS
6
5
4
3
2
1
Wet Well
Oil House
Coal Storage Shed
Azimuth of Base line stone 1000 to Stone
Pit Pump Shaft
Tool House
Drawing Conduit Chamber
Drawing Conduit
South Line of City Property
24 Sewer
Shop
Wiggin's Ferry
Incline
Inlet Tower
MISSISSIPPI RIVER
Homer's Dike
ST. LOUIS WATER WORKS
LOW SERVICE EXTENSION
MAP OF
LOW SERVICE WORKS
AT CHAIN OF ROCKS.

FIG. 1.

alumina. This plan necessitated the cutting down of the walls separating the basins so as to form a series of weirs over which the water would flow in a thin sheet in passing from basin to basin. The weirs were 610 ft. in length, surfaced with brick, with baffles, for agitating and aerating the water, made by setting one course of brick on end. At the north end of the basins a receiving chamber, 610 ft. long, was built to distribute the water, at its entrance into the system of basins, over a weir, similar to the weirs between the basins. This chamber was separated into two longitudinal compartments by a middle wall, perforated at the bottom with numerous openings about 3 in. high by 24 in. long. The water coming into the west end of the northern compartment, filled the southern compartment through the openings at the base of the middle wall and flowed evenly over the weir into the first basin. This chamber was designed with curved sides and bottom so that sediment would not accumulate and stop the passage of water through the small openings. In actual service this chamber admirably fulfilled the purposes for which it was designed, but, for reasons which developed later, its use was discontinued.

The original idea was to use sulphate of alumina as a coagulant, and to introduce it, in the form of a solution, along the weir between the first and second basin, through a submerged pipe lying along the upper side of the weir and discharging through small openings 2 or 3 ft. apart. A similar scheme for introducing a solution of sulphate of alumina into water for the purpose of aiding sedimentation was and had been in use at the Kansas City Water-works for several years.

The problem of removing the sediment from the basins was to be solved by using a floating dredge, designed somewhat after the models of those belonging to the Mississippi River Commission. The dredge had been built and used experimentally in the basins a year or so before this time.

Some time in August, 1903, the attention of the Water Commissioner was called to the use of lime and sulphate of iron as coagulants in conjunction with mechanical filters at Quincy, Ill. All available information on the subject was collected, and in October, 1903, experiments were started in the Laboratory of the Water Department to determine the relative efficiency and cost of the lime and sulphate-of-iron treatment with similar results produced by sulphate of alumina. In

a very short time there seemed to be no question of the superiority of the lime and sulphate-of-iron treatment in cost, efficiency and safety.

In November, 1903, the Water Commissioner and the writer visited the Quincy plant and were shown the practical application of the lime and sulphate-of-iron process in all its details. The action of these coagulants on the raw water on a large scale, was so exactly a reproduction of the laboratory work, that the conclusion was inevitable that this process, modified to suit St. Louis conditions, would produce a wonderful improvement in the water supply.

Work was immediately begun on designing methods of application, of storing and handling chemicals, of changing projected plans to conform to later ideas, and of abandoning certain work already done. On March 22d, 1904, the plant was put in operation, and St. Louis has had no muddy water since, except on the few occasions when the operation of the plant has been interfered with by breakdown or accident.

The treatment consists in the addition of sulphate of iron and lime to the water in such quantities as are necessary to produce an efficient coagulation and a rapid subsidence of all suspended matter. The amount of sulphate of iron added varies from ½ gr. to 4 gr. per gal. of raw water, and the lime from 5 to 9 gr., depending on the quality of the raw water.

The Mississippi River water holds in solution bi-carbonate of lime and magnesia, free carbonic acid and sulphate of lime, all of which are reduced by the treatment except the sulphate of lime which is slightly increased on account of the action of the lime upon the sulphate of iron as follows:

$$Fe\,SO_4 + 7\,H_2O + CaH_2\,(CO_3)_2 = Fe\,H_2\,(CO_3)_2 + CaSO_4 + 7\,H_2O$$

The action of the lime is then as follows:

$$Fe\,H_2\,(CO_3)_2 + 2\,Ca\,(OH)_2 = Fe\,(OH)_2 + 2\,CaCO_3 + 2\,H_2O$$
$$CO_2 + Ca\,(OH)_2 = CaCO_3 + H_2O$$
$$Ca\,H_2\,(CO_3)_2 + Ca\,(OH)_2 = 2\,CaCO_3 + 2\,H_2O$$
$$Mg\,H_2\,(CO_3)_2 + 2\,Ca\,(OH)_2 = Mg\,(OH)_2\,CaCO_3 + 2\,H_2O$$

The ferrous hydrate [$Fe\,(OH)_2$] remaining in the water oxidizes into ferric hydrate, thus:

$$2\,Fe\,(OH)_2 + H_2O + O = Fe_2\,(OH)_6.$$

The normal carbonate of lime ($CaCO_3$) is only slightly soluble in water, and crystallizes about the particles of suspended matter and on the flocculent precipitate of ferric hydrate, which also entangles portions of the suspended matter, and serves to drag down the whole mass of matter in suspension, together with a very large percentage of the bacteria.

The hydrate of magnesia [$Mg(OH)_2$], precipitated, will settle out of the water slowly. It is insoluble, but as the water absorbs carbonic acid from the air, it will gradually be converted back into bi-carbonate of magnesia and again be taken into solution.

The original methods of applying the lime and sulphate of iron to the water, which have been considerably modified during the three years' operation, were the result of careful study and observation.

At Quincy, Ill., and at other places, the lime was reduced to a solution of lime-water and added in a constant quantity of known strength. To undertake to add the requisite quantity of lime, in the form of lime-water, to the raw water used in St. Louis, meant the reduction of from 30 to 40 tons of lime daily, which would necessitate a maximum capacity of about 10 000 000 gal. of lime-water. To manufacture such a quantity of lime-water every 24 hours would require an equipment of machinery and storage so large as to make the idea utterly impracticable, besides introducing other complications on account of the variations in strength of the lime-water due to temperature, which would require accurate and frequent determinations and changes in the measured quantities applied to the raw water. After a great many experiments it was decided to apply the lime in the form of milk of lime carrying from 5 000 or 6 000 gr. of lime per gal.

The method of manufacturing the milk of lime was as follows: Circular tanks, 6 ft. in diameter and 3 ft. deep, with revolving rakes to keep the hydrate of lime in suspension, were provided, and found to be capable of slaking 1 800 lb. of lime per hour each, when supplied with about 30 gal. of water per minute at a temperature of 120° fahr. The lime was added in weighed quantities to these tanks at 5-min. intervals, the weights corresponding to the number of gallons pumped per 5 min. multiplied by the number of grains per gallon added. This arrangement gave a continuous flow of milk of lime of practically constant strength, so long as the weight of lime added every 5 min. remained constant.

The tanks for dissolving the sulphate of iron were 4 by 5 ft. and 3 ft. deep, supplied with a continuous flow of cold water introduced at the bottom of the tanks through a number of ⅜-in. pipes with perforated caps, giving a network of horizontal streams over the bottom of the tank. The sulphate of iron was dumped into the tanks at 5-min. intervals, in weighed quantities corresponding to the gallons of water being pumped and the grains per gallon added.

The quantities of water supplied to the lime and sulphate-of-iron tanks may vary considerably, provided that so much water is not added as to produce a velocity high enough to carry small particles over in suspension, and also that, in the first case, enough water is supplied to each tank to convert the lime into a milk of lime which will flow readily, and, in the other, to dissolve the sulphate of iron entirely, since the proper weights of lime and sulphate of iron are added at 5-min. intervals.

The solution of iron sulphate is introduced into the water supply through an iron pipe leading down the uptake shaft, where it becomes thoroughly mixed with the river water in its passage to and through the pumps.

The milk of lime flows from the mixing tanks to a collecting tank, where it is diluted and cooled to about 100° fahr., whence it is pumped by a 20-h.p. centrifugal pump to the delivery well which receives the discharge from the low-service pumps. The milk of lime falling into this delivery well is mixed thoroughly with the water discharged from the pumps, which has already been treated with the sulphate of iron. By the time the water reaches the basin, coagulation has taken place, and the settlement begins as soon as the initial velocity of the entering water is sufficiently reduced. Fig. 2, Plate XXIV, shows within what limited space this occurs.

Probably more than 90% of the suspended matter settles to the bottom of the basin into which the water is first introduced, and the degree of improvement in clarification produced by the journey of the water through the succeeding basins is not so marked, yet there is a vast difference between the quality of the water in the second basin and that in the sixth and last.

A long series of experiments was made in the attempt to determine the velocity of the water through the basins. The velocities are so low and the variations so great, the direction of currents so indeterminate

and so readily influenced by the wind, that it has been impossible to do more than to approximate roughly the velocities along the center line of the basins, the weather conditions being practically the same for all observations.

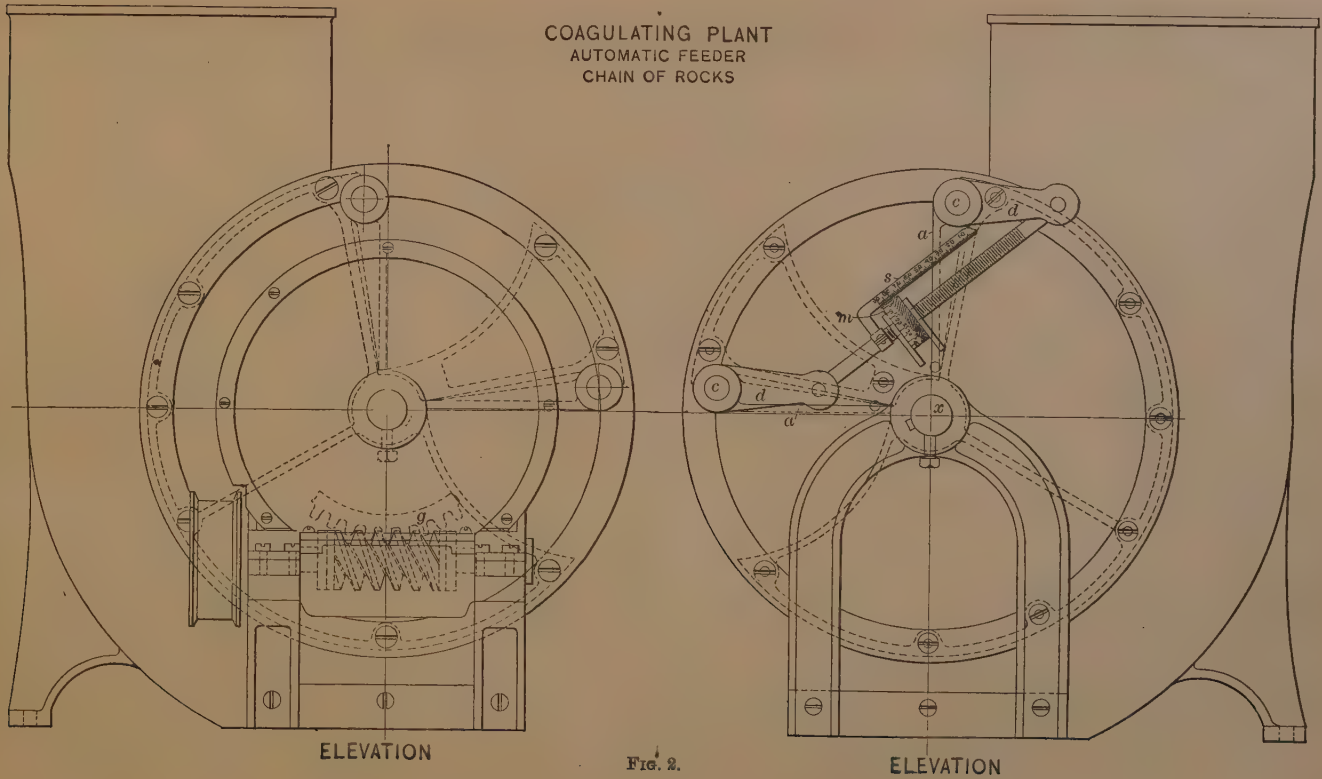
The following velocities are averaged from a number of observations, rod floats being used, the quantity of water passing through the basin at the time being at the rate of 70 000 000 gal. per 24 hours:

Float	submerged	6	in.,	velocity	5.35	ft.	per	min.
"	"	1	ft.	"	5.60	"	"	"
"	"	2	"	"	5.57	"	"	"
"	"	3	"	"	5.06	"	"	"
"	"	4	"	"	4.86	"	"	"
"	"	5	"	"	3.78	"	"	"
"	"	6	"	"	3.33	"	"	"
"	"	7	"	"	2.92	"	"	"
"	"	8	"	"	2.65	"	"	"
"	"	9	"	"	2.37	"	"	"
"	"	10	"	"	2.35	"	"	"

Floats of various designs were experimented with, for example, spherical surface floats of wood and of copper, double-ball floats, double floats with the upper float hemispherical and the lower one spherical, the two connected in one case by cotton cord, again by thin wire, and again by a small wooden rod, besides other forms and combinations. Fairly uniform results were obtained from the rod floats made of wood, 1§ in. in diameter. Observations taken by all other floats were so erratic that no use could be made of them.

Soon after this clarification process was put into operation, it was discovered that the agitation produced by the water flowing over the weirs was interfering with the sedimentation. The interference with the smooth flow of the water tended to break up the coagulated particles and to reduce them to such a state of fineness that sedimentation would not occur much more readily than in the raw water. This caused the abandonment of the use of the entrance chamber at the north end of the basins, and led to the practice of submerging all the weirs, so that the flow from basin to basin was smooth and uninterrupted. The water was introduced into Basin No. 1 through the gate connected with the filling conduit on the west side of that basin. These changes produced

COAGULATING PLANT
AUTOMATIC FEEDER
CHAIN OF ROCKS



ELEVATION

ELEVATION

FIG. 2.

excellent results until the sediment accumulated in Basin No. 1 to such an extent that the inflow of water stirred up the deposited sediment and actually remuddied the clear water. To correct this, a baffle was built in front of the inflow pipes of Basins No. 1 and No. 6. This baffle brought all the incoming water to the surface and kept the currents at or near the surface. With this arrangement the basin could be kept in service until the sediment accumulated within a few inches of the top of the baffle. By opening the sewer gate on the east side of the basin, and leaving it open until the effluent became almost clear, a vast quantity of mud was washed out of the basin daily, thus allowing it to be kept in service for much longer periods without being emptied and cleaned. When it finally became necessary to empty and clean Basin No. 1, the water was introduced from the filling conduit into Basin No. 6 through the west gate, reversing the flow through the basin and drawing the clear water from Basin No. 2, while Basin No. 1 was emptied and cleaned. Basin No. 1 was then filled with clear water over the weir from Basin No. 2, and the clear water supply was drawn from Basin No. 1 until Basin No. 6 had to be cleaned, when the flow was again reversed and the operation resumed as in the first case. It soon became apparent that only these two basins would need frequent cleaning under this system of operation, whereas formerly all the basins had to be cleaned regularly, necessitating the constant employment of a force of men in the basins for about seven months of the year. It has developed that these end basins require cleaning after two or three months' service, depending upon the character of the river water. The cost of cleaning is not more than one-third of what it was under the old system, although all the sediment plus the lime and iron sulphate is deposited in the basin now, while 75 or 80% only was deposited there under the old system.

While the practical results obtained by this process of water purification have been very gratifying during the entire time it has been used, it is only within the last year of its operation that the methods, the working, and the results have been subjected to systematic study and analysis. Previous to that time the Department was kept fully occupied in making changes to rectify glaring faults which required neither study nor science to discover, although the remedy was not always so obvious.

Practically all the chemical and bacterial work on the raw and

treated water during the first two years of operation was done outside of the Department by the City Chemist and the City Bacteriologist, and under the direction of the Board of Health.

TABLE 1.—Average Percentages and Reduction by Months.

Date.	Suspended Solids, in parts per million.		Dissolved Solids, in parts per million.		Bacteria, per Cubic Centimeter.		
	River.	Tap.	River.	Tap.	River.	Tap.	Percentage of removal.
April, 1904	2 400	130	189	160	126 500	22 940	86.0
May, 1904	1 548	65	206	161	57 000	5 850	93.0
June, 1904	3 372	39	206	165	27 000	1 050	96.0
July, 1904	2 483	48	212	144	26 000	323	98.7
August, 1904	1 290	40	257	170	15 180	817	95.0
September, 1904	1 000	15	228	177	14 200	225	98.2
October, 1904	460	13	301	194			
November, 1904	228	9	320	195	24 300	4 050	82.5
December, 1904	256	9	352	227	48 250	4 587	90.5
January, 1905	79	16	414	242	20 000	1 857	90.7
February, 1905	29	4	406	280	39 350	1 341	96.5
March, 1905	942	9	225	180	33 700	1 910	94.4
April, 1905	440	6	206	156	66 080	2 200	96.8
May, 1905	965	30	273	234	15 130	900	94.0
June, 1905	1 040	15	231	158	32 310	619	98.4
July, 1905	2 606	41	233	182	91 000	1 750	98.2
August, 1905	1 746	24	216	165	40 400	400	99.0
September, 1905	1 125	16	218	185	32 400	350	98.9
October, 1905	730	15	224	156	53 820	390	99.3
November, 1905	395	10	248	155	46 250	1 863	96.0
December, 1905	388	10	316	186	48 520	848	98.4
January, 1906	496	7	234	151	144 420	1 810	98.7
February, 1906	259	0	247	144	47 220	1 164	97.5
March, 1906	858	3	166	118	66 560	202	99.7

Special tests, taking daily samples for from 10 days to 2 weeks, for the purpose of determining the presence of *B. Coli Communis* have been made at various times and by different observers. Except in one or two doubtful cases, the total absence of *B. Coli* in the tap water was established beyond any reasonable doubt.

Table 2 shows the improvement in the purity of the water supply in a marked degree.

The population of St. Louis, in 1900, was 575 000, in round numbers; in 1907, it is 700 000. The record for 1904 must be read, bearing in mind that the purification process was not started until April and that 1904 was the year of the World's Fair in St. Louis, which temporarily increased the population by many thousands, and, under the conditions prevailing in previous years, an increase in the number of cases and deaths from all diseases could reasonably have been expected.

TABLE 2.—Cases and Deaths from Typhoid Fever in St. Louis, since January 1st, 1900.

	1900.		1901.		1902.		1903.		1904.		1905.		1906.		1907.	
Months.	C.	D.	C.	D.	C.	D.	C.	D.	C.	D.	C.	D.	C.	D.	C.	D
January	57	19	76	8	103	14	81	15	65	19	20	5	28	7	12	5
February	42	5	55	14	52	16	55	15	37	7	14	6	17	5	15	6
March	117	16	32	11	34	11	91	20	61	20	19	4	18	6	14	2
April	34	4	24	4	48	14	79	17	38	12	18	1	17	5	....	....
May	29	4	32	8	38	11	86	15	56	18	32	9	12	3	....	....
June	26	8	26	6	28	7	90	16	25	6	15	6	14	3	....	....
July	43	15	49	13	83	14	189	26	65	23	59	18	50	8	....	....
August	119	20	185	31	88	26	221	37	178	30	130	24	154	16	....	....
September	253	13	138	24	169	21	261	42	180	36	163	17	125	29	....	....
October	262	30	205	32	194	32	183	30	99	24	105	23	101	14	....	....
November	127	15	154	27	169	31	131	23	44	17	63	7	50	11	....	....
December	104	19	125	20	106	25	119	31	24	13	36	6	24	5	....	...
Totals	1 213	168	1 101	198	1 112	222	1 586	287	872	225	674	126	610	112	....	....

The typhoid rate was 29.4 per 100 000 in 1900, increasing to more than 40 per 100 000 in 1903, and then decreasing to 16 per 100 000 in 1906. This, in itself, does not express the total beneficial effect on the general health arising from the use of pure water. Allen Hazen, M. Am. Soc. C. E., estimates that where one death from typhoid has been avoided through an improved water supply, probably two or three deaths from other diseases have been avoided. The following record seems to indicate a higher proportion of decrease than Mr. Hazen's estimate would give:

Annual Mortality Record, St. Louis, Mo., 1900 to 1906, from Disease Alone:

1900	9 217	1904	10 695
1901	9 916	1905	9 545
1902	9 654	1906	9 214
1903	10 320		

The original method of preparing both the iron solution and the milk of lime was not entirely satisfactory from the first. The iron solution, as it came from the tank, varied in strength at times as much as 25% from the normal, due to adding the charges at 5-min. intervals while the stream of water flowing through the mass was constant. Evidently as the mass dissolved toward the end of the 5-min. period, less

sulphate of iron would be exposed to the water, and the solution would be weaker. The extent of variation could be limited by reducing the water supply to the minimum quantity necessary to keep the sulphate dissolved approximately as fast as added, but a simpler and more certain way of maintaining a constant-strength solution would be by feeding the sulphate to the tank continuously and evenly. The sulphate of iron is shipped in bulk, and is known as "sugar" sulphate. It is similar in fineness to granulated sugar, and when perfectly dry will flow like dry sand. The physical condition of the sulphate is not uniform enough to allow of the use of a standardized orifice, so that it became necessary to design an apparatus for automatic force feeding, which would ensure the measurement of equal volumes, in equal times, and be susceptible of adjustment to various rates of feeding. The range of the apparatus must cover all rates from $2\frac{1}{2}$ to 50 lb. per min.

The machine shown by Fig. 2 was designed and built in the Department, and, when operated, its measurements were found to be absolutely uniform. The machine consists of a cylinder about 12 in. in diameter and 5 in. long, with two pockets the size of which may be varied by the movement of the side, *a*, which revolves about *c* as a center. This adjustable side fits closely to the ends of the cylinder and to the curved side of the pocket. These adjustable sides are moved by the arms, *d*, *d*, which, in their turn, are moved by the micrometer screw, *m*, while the positions of the movable sides are shown by the scale, *s*. The cylinder revolves on the axis, *x*, and is driven by the worm gearing, *g*. The cylinder is partially covered by a casing which serves to hold the full load in the pockets until they reach the position for dumping, and also to prevent the escape of material from the hopper. The cylinder revolves at a constant speed, about 12 rev. per min., and measures and discharges a constant volume of material in equal periods of time.

Tests made to determine the available amount of calcium hydrate carried by the milk of lime, as manufactured by the system of tanks described heretofore, showed that there occurred a heavy loss, amounting at times to as much as 35% of the theoretical value of the quicklime weighed into the tanks. About 3% of this loss is due to impurities in the quicklime, such as sand, dirt, flint and underburned stone. Another portion of loss is due to the air-slaking of the quicklime

in transit and in storage. In very warm, moist weather this loss is considerable, but during the greater part of the year it is very small. The greatest part of the loss was found to be due to imperfect hydration, small particles of unslaked lime being carried out of the tanks in suspension. Lowering the velocity through the tanks by reducing the quantity of water, at 120° fahr., supplied to them, increased the efficiency and raised the temperature of the tank contents. Increasing the temperature of the water supplied to the tanks from 120° to 150° fahr., without cutting down the quantity, resulted in an increased efficiency of about 15%, and raised the temperature in the tanks to almost 200° fahr.

By supplying the tanks with a quantity of water (at 50° fahr.) approximately equal by weight to three times the weight of quicklime added, complete hydration resulted, with a tank temperature of more than 200° fahr. This was unsatisfactory because the tank contents became too thick to flow readily through the outlet pipe, and, if not constantly watched, would eventually become so stiff as to stop the stirring apparatus. An addition of too much cold water would reduce the temperature of the tank contents too much for efficient hydration. This led to the idea of utilizing the heat of the effluent from the tanks for heating the water supply. This was accomplished by the addition of an auxiliary tank into which the milk of lime is conducted from the several slaking tanks. This auxiliary tank contains a coil through which the cold water flows on its way to a head tank, from which the supply is piped to the slaking tanks. The head tank is equipped with a float valve to regulate the flow of incoming water, so that a constant head is kept on all the feed pipes leading to the slaking tanks. Each feed pipe has, at its lower end, an adjustable orifice to regulate the quantity of water supplied to each tank, so that the quantity of water entering each tank is from 3½ to 3¾ times as much by weight as the lime added to that tank. The temperature of the tank contents is more than 200° fahr., and that of the water in the head tank about 100° fahr., varying with the temperature and quantity of cold water entering the coils. This arrangement has been in service for several months, and has shown a decided reduction in the expense of operation.

The total quantities of sulphate of iron and lime used during the first year of operation were: 3 578 tons of sulphate of iron, and 14 291 tons of lime; during the second year: 4 138 tons of sulphate of iron,

and 11 814 tons of lime; during the third year: 4 050 tons of sulphate of iron, and 14 081 tons of lime.

The total quantity of water treated during the first year was 33 133 million gal.; during the second year, 26 334 million gal.; and during the third year 26 682 million gal.

The average daily consumption of water has not increased proportionately with the population since the clarification of the water supply.

The population of the city has increased from 575 000 in 1900 to 700 000 in 1907. The average daily water consumption for each year during this period follows:

1900-01	63 000 000 gal.
1901-02	67 173 000 "
1902-03	66 211 000 "
1903-04	69 916 000 "
1904-05	79 052 000 "
1905-06	69 000 000 "
1906-07	70 109 000 "

The average quantities of sulphate of iron and lime used during the first year were 1.5 gr. of sulphate of iron and 6.0 gr. of lime per gallon of water treated; during the second year, 2.2 gr. of sulphate of iron and 6.28 gr. of lime; during the third year, 2.13 gr. of sulphate of iron and 7.39 gr. of lime.

The average cost per 1 000 000 gal. of water treated (including labor and power) for the first year was $3.60, for the second year, $3.99 and for the third year, $4.62. The cost in detail is as follows:

	1904-05	1905-06	1906-07
Cost of lime	$1.89	$1.74	$2.45
Cost of sulphate of iron	1.07	1.53	1.44
Labor	0.57	0.62	0.58
Power	0.07	0.09	0.07
Repairs	....	0.01	0.08
	$3.60	$3.99	$4.62

The gradual increase in the cost of purification is due to several causes. During the first year the work was conducted cautiously, and

quantities (both of sulphate of iron and of lime), too small to produce efficient coagulation and settlement were often used. The water as drawn from the settling basins was often turbid and nearly always cloudy. It was not until the latter part of the first year that the efficiency of the treatment began to approach the results which were obtained during the third year. There was also the fear of having caustic lime in the effluent, and, at that time, no definite method of determining the quantity of lime which it would be safe to add had been devised; so that, in trying to avoid the error of using too great a quantity, it was natural to fall into the mistake of adding too little. The increase in cost per 1 000 000 gal. during the third year was due principally to the fact that the price of lime was raised from $3.88 to $4.65 per ton. The average quantity of lime added, in grains per gallon, was greater during the third year on account of the longer period of very turbid water in the river, and because of greater accuracy in proportioning the proper quantity of lime necessary each day and a consequent production of a continuous supply of pure and clear water.

Tables 3 and 4 give comparative analyses of the river and treated water, showing the reduction of bacteria and tests for *B. Coli Communis*. The bacterial tests were not started until November, 1906, on account of delay in equipping the laboratory with the necessary apparatus.

The maximum quantity of sulphate of iron added to the water on any occasion during the third year was 3.75 gr., the minimum being 1.25 gr. per gal. of water treated; of lime, the maximum was 11 gr., and the minimum, 3 gr. per gal. of water treated. These figures are calculated on the weights of sulphate of iron and lime as taken from the bins, but, in the case of the lime, the actual efficient quantity is much less than the record shows, on account of deterioration in transit and in storage from air-slaking; for example, in the case of the 11 gr. mentioned, the actual amount of efficient lime, as determined by analysis, was 7.7 gr. This explains why the average for the year, in grains per gallon, is as high as 7.39; the average of efficient lime added would be considerably lower. The sulphate of iron does not deteriorate appreciably in storage.

The alkalinity of the treated water during the third year varied from 35 to 75 parts per million, the average being 49. The increase of incrustants through the action of the sulphate of iron averaged 13

TABLE 3.—Analyses of River Water, St. Louis, Mo.

Date.	Turbidity.	Color.	Suspended Solids.	Dissolved Solids.	Calcium Ions.	Magnesium Ions.	Sulphate Ions.	Alkalinity.	Incrustants.	Neutral Carbonates.	CO_2	Bacteria, per cubic centimeter.
1906.												
Apr. 6..	1 650	45	.....	.....	33	13	19	90	22	....	1	.....
" 13..	1 000	59	.....	.....	28	10	12	106	.....	.....	2	.....
" 20..	1 800	51	1 116	210	36	12.6	20	120	22	.....	3	.....
" 27..	500	45	319	189	30	9.5	30	112	.....	.....	2	.....
May 4..	180	51	108	150	32	8.3	25.3	98	17	.....	1	.....
" 11..	1 200	35	671	206	41	10 4	33	126	22	.....	2	.....
" 18..	800	35	1 008	246	52	12	41	143	36	.....	2	.....
" 25..	800	31	711	289	45	15	53	148	28	.....	1	.....
June 1..	1 050	35	842	281	45	11.1	44	140	20	.....	1	.....
" 8..	2 400	31	2 090	264	46	11.5	74	134	28	.....	3	.....
" 15..	3 000	31	3 910	263	42	11.5	74	137	16	.....	3	.....
" 22..	3 600	35	5 030	273	41	10.8	69	133	15	.....	2	.....
" 29..	3 600	31	4 220	291	44	13.4	80	134	32	.....	4	.....
July 6..	3 000	35	3 402	263	.....	.....	.....	128	.....	.....	3	.....
" 13..	2 400	41	2 084	244	....	.....	.....	118	.....	.....	2	.....
" 20..	1 500	35	1 673	246	.....	.....	.....	120	.....	.....	2	.....
" 27..	1 500	41	1 493	218	35	12	76	128	20	.....	1	.....
Aug. 3..	1 500	31	1 267	272	44	13	80	129	34	12	....	.....
" 10..	1 350	35	966	238	.....	.....	.....	130	.....	.....	....	.....
" 17..	1 500	35	1 225	213	.....	.....	.....	130	.....	.....	....	.....
" 31..	1 800	35	1 654	218	.....	.....	.....	130	.....	2	....	.....
Sept. 7..	1 350	40	850	243	42	12	55	134	20	10	....	.....
" 14..	900	35	720	223	.....	.....	.....	137	.....	6	....	.....
" 21..	900	35	627	246	.....	.....	.....	143	.....	14	....	.....
" 28..	2 400	35	1 367	225	.....	.....	.....	135	.....	.....	1	.....
Oct. 5..	1 050	41	849	201	37	12.5	47	119	25	.....	2	.....
" 12..	900	35	750	248	45	13.6	56	141	29	.....	1	.....
" 19..	900	35	670	293	.....	.....	.....	152	.....	.....	2	.....
" 26..	550	35	412	314	.....	.....	.....	165	.....	8	....	.....
Nov. 2..	700	35	771	313	56	16.6	71	174	36	14	....	26 900
" 9..	900	41	558	294	54	17	69	166	40	8	....	35 400
" 16..	750	59	497	288	.....	.....	.....	158	.....	10	....	42 000
" 23..	500	63	444	284	.....	.....	.....	156	.....	12	....	32 300
" 30..	400	71	371	290	.....	.....	.....	165	.....	12	....	20 700
Dec. 7..	450	65	362	280	48	17.8	43	165	28	12	....	.....
" 14..	300	45	231	304	52	16.3	48	170	28	10	....	.....
1907.												
Jan. 11..	450	35	354	296	53	14.6	48	164	38	.....	....	25 900
" 18..	600	60	1 269	219	34	15.8	38	120	31	.....	2	20 400
" 25..	1 800	45	1 817	138	.....	.....	.....	67	.....	6	....	87 800
Feb. 1..	500	55	630	169	24	9.7	25	90	10	0	2	46 600
" 8..	250	70	212	220	38	15	48	135	11	6	4	25 800
" 15..	200	35	386	246	.....	.....	.....	168	.....	.....	3	18 200
" 21..	800	35	492	290	.....	.....	.....	158	.....	.....	3	.....
Mar. 1..	1 500	45	1 700	230	42	11	62	140	10	.....	5	85 809
" 8..	900	45	1 088	248	40	15	48	132	31	.....	4	48 700
" 15..	1 500	70	2 006	216	34	12	51	110	24	.....	2	88 200
" 22..	1 500	50	1 905	275	.....	.....	.....	146	.....	.....	3	26 100
" 29..	1 200	30	1 190	267	.....	.....	.....	142	.....	.....	2	12 900

TABLE 4.—WATER ANALYSES, CLEAR WELL AT BISSELL'S POINT.

DATE.	Color.	Suspended Solids.	Dissolved Solids.	Calcium Ions.	Magnesium Ions.	Sulphate Ions.	Alkalinity.	Incrustants.	Caustic Alkalinity.	Bacteria, per cubic centimeter.	Removal, per cent.
1906.											
April 6	20	.....	.....	25	5.8	22	37	48	5	.....	.....
" 13	10	.....	.....	24	3.6	55	37	37	3	.....	.....
" 20	12	8.6	150	20	2.9	55	40	21	0	.....	.....
" 27	12	5.2	146	28	4.1	46	42	45	6	.....	.....
May 4	10	10.0	110	22	5.0	32	46	29	7	.....	.....
" 11	12	27.2	151	24	3.8	69	40	36	6	.....	.....
" 18	12	10.0	155	25	4.3	60	43	37	1	.....	.....
" 25	14	7.6	190	21	6.4	64	44	40	0	.....	.....
June 1	10	0.0	160	22	5.0	44	39	36	1	.....	.....
" 8	12	5.6	184	26	3.8	87	39	42	3	.....	.....
" 15	12	5.2	201	31	2.3	83	40	46	6	.....	.....
" 22	10	4.4	203	36	1.7	83	45	52	15	.....	.....
" 29	10	6.0	246	41	1.2	90	52	54	20	.....	.....
July 6	10	5.8	263	.....	.....	.....	51	.....	15	.....	.....
" 13	10	6.4	215	.....	.....	.....	53	.....	15	.....	.....
" 20	10	4.4	212	.....	.....	.....	55	.....	21	.....	.....
" 27	12	5.6	187	25	4.8	80	47	34	3	.....	.....
Aug. 3	12	2.4	192	23	6.0	87	48	35	0	.....	.....
" 10	10	0.0	180	.....	.....	.....	41	.....	0	.....	.....
" 17	10	2.4	160	.....	.....	.....	.....	.....	0	.....	.....
" 31	10	7.6	147	.....	.....	.....	45	.....	1	.....	.....
Sept. 7	10	3.6	159	18	4.8	66	46	19	0	.....	.....
" 14	10	1.2	168	.....	.....	.....	47	.....	5	.....	.....
" 21	10	1.6	169	.....	.....	.....	50	.....	2	.....	.....
" 28	12	3.6	171	.....	.....	.....	50	.....	0	.....	.....
Oct. 5	15	3.6	155	25	5.8	56	52	34	4	.....	.....
" 12	10	3.6	170	22	7.2	80	53	32	0	.....	.....
" 19	10	3.2	182	.....	.....	.....	59	.....	0	.....	.....
" 26	10	4.0	194	.....	.....	.....	57	.....	1	.....	.....
Nov. 2	10	2.8	206	21	12.5	77	59	45	0	2 600	90.0
" 9	12	1.6	209	21	11.8	71	56	46	2	2 000	94.3
" 16	10	3.2	196	.....	.....	.....	55	.....	0	5 000	85.7
" 23	10	0.4	196	.....	.....	.....	58	.....	0	600	98.1
" 30	10	4.0	197	.....	.....	.....	60	.....	2	1 200	94.2
Dec. 7	8	5.6	192	21	14.5	53	60	52	0	.....	.....
" 14	10	4.4	186	20	13.0	48	58	47	0	.....	.....
" 21	8	3.2	194	.....	.....	.....	62	.....	0	.....	.....
" 28	8	0.0	210	.....	.....	.....	68	.....	0	.....	.....
1907.											
Jan. 4	8	0.0	222	.....	.....	.....	75	.....	0	.....	.....
" 11	8	6.0	193	21	12.5	56	64	39	0	330	98.7
" 18	8	1.2	191	22	15.0	48	56	60	0	935	95.4
" 25	12	0.0	135	.....	.....	.....	51	.....	17	20	99.9
Feb. 1	12	8.4	130	22	7.9	33	48	37	0	1 200	97.4
" 8	12	1.2	138	21	10.0	55	52	34	0	160	99.4
" 15	12	1.6	148	.....	.....	.....	56	.....	0	.....	.....
" 21	10	0.8	181	.....	.....	.....	61	.....	0	.....	.....
Mar. 1	10	2.0	179	21	10.0	73	51	45	11	00	100.0
" 8	12	4.8	171	19	11.0	64	43	59	0	50	99.9
" 15	12	4.8	165	20	11.0	66	41	56	0	40	99.9
" 22	10	0.4	175	.....	.....	.....	39	.....	1	50	99.8
" 29	10	0.0	181	.....	.....	.....	43	.....	0	40	99.7

FIG. 1.—RECEIVING CHAMBER, NORTH SIDE OF BASIN NO. 1, AS ORIGINALLY BUILT.

FIG. 2.—RECEIVING CHAMBER REMODELED TO INTRODUCE WATER INTO BASIN NO. 1 THROUGH FOUR 3 BY 3-FT. GATES.

parts per million, calculated as calcium carbonate. The softening actually accomplished by the treatment is a reduction of the lime carbonate to 53% of that in the raw water, and of the magnesia to 57%; the average of 204 determinations each, of the amounts of calcium and magnesium in the raw water, were 42.3 and 13.1 parts per million, respectively; in the treated water, 22.8 and 7.54 parts per million.

TABLE 5.—Percentage of Reduction in Bacteria as the Water Passes Through the Basins:

	Basin No. 1.	Basin No. 2.	Basin No. 3.
October, 1906	76.4%	82.8%	.....
November, 1906	89.0%	90.0%	90.1%
December, 1906	72.2%	77.5%	67.8%
January, 1907	93.7%	96.0%	97.4%
February, 1907	96.0%	98.4%	99.0%
March, 1907	96.1%	99.1%	99.8%

During each day, as often as conditions require, samples are taken from each basin, and the caustic alkalinity of each sample determined by titration. Throughout the basins the caustic alkalinity is kept as low as is consistent with efficient coagulation, the aim being to keep down the charge of lime until the last basin shows little or no caustic alkalinity. It is not possible to maintain any uniform set of conditions throughout the basins, on account of the great and sudden fluctuations in the quality of the river water. The quantity of lime added being controlled in this way, the changes in the amount of sulphate of iron necessary are determined from the appearance of the coagulation and from the condition of the water in each basin, turbidity determinations being made from a portion of the samples taken for testing the caustic alkalinity.

Determinations of the turbidity, color, etc., of the raw water and of the water in the basins are carried on daily, a record being kept, as shown in Table 6.

The coagulating plant, which has been in use for more than three years, was originally built as a temporary affair, with the expectation that it would be replaced by a permanent structure after the World's Fair in 1904. The cost of this temporary coagulating plant, with all machinery and connections, was less than $10 000. The cost of changing the settling basins, necessary to put in operation the purification scheme, amounted to about $25 000.

TABLE 6.—Laboratory Record, Chain of Rocks, St. Louis, Mo.

1907.		River Water.						Grains, per Gallon.		Entrance.		Basin No. 1.		Basin No. 2.		Basin No. 3.		Basin No. 4.		Basin No. 5.		Basin No. 6.		Wind.	
April.																									
Day.	Hour.	Stage.	Temperature.	Turbidity.	Color.	CO_2	Alkalinity.	Lime.	Iron.	Color.	Caustic.	Color.	Caustic.	Color.	Caustic.	Color.	Caustic.	Color.	Caustic.	Color.	Caustic.	Color.	Caustic.	Direction.	
1	8 A. M.	88.9	55°	1 200	35	3	132	6.5	2.25	14	8	14	7	12	8	12	5	12	2	12	3	10	4	N. E.	Mild.
	9 "	.....	.....	.....	....	...	.....	6.0	2.0	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	10:30 "	.....	.....	.....	....	...	.....	6.0	2.0	14	2	14	4	12	5	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	1 P. M.	.....	.....	.....	....	...	.....	6.0	2.0	14	1	14	2	12	4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	3:30 "	.....	.....	.....	....	...	.....	6.0	2.0	16	2	14	2	14	1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
2	8 A. M.	89.2	54°	1 500	45	3	132	6.25	2.25	16	4	14	0	14	—1	14	—1	12	1	12	1	12	1	S.	Brisk.
	9 "	.....	.....	.....	....	...	.....	6.75	2.25	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
3	8 "	89.2	55°	1 500	45	3	128	6.75	2.25	14	4	14	4	12	5	12	4	12	2	12	1	10	1	S.	Brisk.
	10 "	.....	.....	.....	....	...	.....	6.75	2.25	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	3:30 P. M.	.....	.....	.....	....	...	.....	6.75	2.25	14	7	14	6	12	4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	8 A. M.	89.0	56°	1 500	35	3	122	6.75	2.25	14	6	14	6	12	7	12	6	12	6	12	5	10	5	W.	Mild.
	3 P. M.	.....	.....	.....	....	...	.....	6.25	2.0	14	1	12	7	12	9	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	8 A. M.	88.9	56°	1 350	35	3	126	6.25	2.0	14	11	14	6	12	9	12	6	12	5	10	1	10	4	N.	Brisk.
	9 "	.....	.....	.....	....	...	.....	6.0	2.0	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	10:30 "	.....	.....	.....	....	...	.....	6.0	2.0	14	8	12	9	12	9	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	3 P. M.	.....	.....	.....	....	...	.....	6.0	2.0	14	8	12	6	12	6	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
6	8 A. M.	88.9	54°	1 350	35	3	128	6.0	2.0	14	5	14	8	14	8	12	6	12	5	12	5	10	4	E.	Brisk.
	9 "	.....	.....	.....	....	...	.....	5.75	2.0	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	10:30 "	.....	.....	.....	....	...	.....	5.75	2.0	14	3	14	5	12	8	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	11:30 "	.....	.....	.....	....	...	.....	6.0	2.0	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	1 P. M.	.....	.....	.....	....	...	.....	6.0	2.0	14	5	14	2	12	5	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
7	8 A. M.	88.9	54°	1 500	35	3	130	6.0	2.0	16	3	14	8	14	8	14	6	12	0	12	6	10	6	S. W.	Brisk.
	9:30 "	.....	.....	.....	....	...	.....	6.0	2.25	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	1 P. M.	.....	.....	.....	....	...	.....	6.25	2.25	14	2	14	1	14	7	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
8	8 A. M.	88.7	55°	1 350	40	3	125	6.25	2.25	14	4	14	5	12	3	12	3	12	2	10	2	10	4	W.	Brisk.

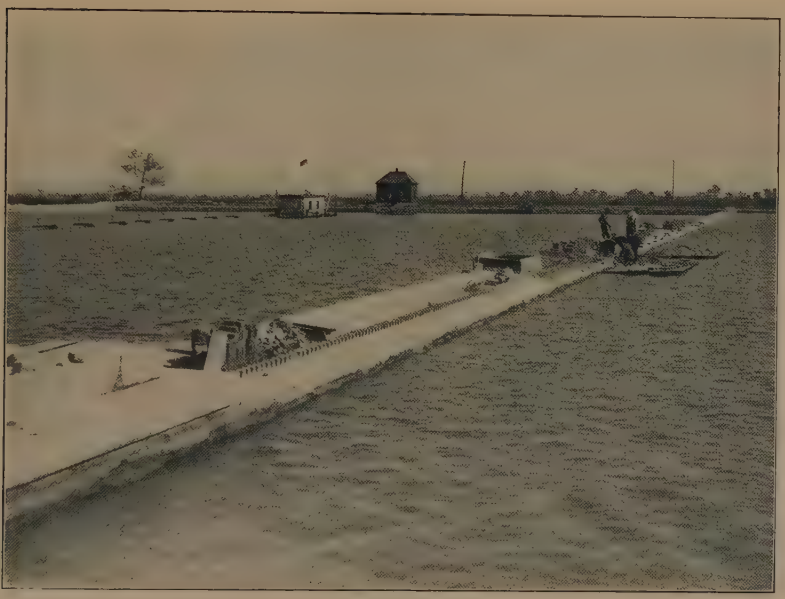


FIG. 1.—WEIR CONSTRUCTION.

FIG. 2.—WATER ENTERING BASIN, SHOWING LINE OF DEMARCATION BETWEEN CLEAR AND MUDDY WATER.

When the process was put into operation, however, defects and new problems presented themselves in rapid succession, and it soon became evident that a design for a permanent plant would have to await the solution of some of these problems. After three years of study and experiment, plans and specifications have been prepared for a fire-proof building, equipped with crushers, conveyors, elevators, feeders, mixing-tanks, etc. The contract for this building was let on June 11th, 1907, for the sum of $89 500, to be completed by January 1st, 1908. Plans, elevations and sections are shown on accompanying plates.

The building will contain eight circular bins, of 20 ft. inside diameter and 40 ft. high, for the storage of sulphate of iron and lime. These bins will have a capacity of 10 600 cu. ft. each, providing for the storage of about 160 days' supply of sulphate of iron and about 45 days' supply of lime, at the present rate of use.

The lime bins will be practically air-tight to prevent deterioration while in storage. In April, 1906, a small bin, 6 by 6 by 6 ft., interior dimensions, was built of wood with double walls, the space between filled with pitch to one-half the height, the upper half being lined with tar paper covered with pitch. This bin was filled with lime, and sealed. A careful watch was kept for signs of heating, but up to June 1st, 1907, no evidence of deterioration had been found. In August, 1906, another bin of the same size was built of concrete 6 in. thick, and filled with lime. No damage has occurred to the lime in this bin after ten months' storage. Both these bins have stood exposed to the weather, without any protection, ever since they were built.

The bins in the new building will be constructed of reinforced concrete, faced on the outside with brick. Sections showing the reinforcement are shown on Plate XV. The central part of the building will be of brick, and divided into three floors, *viz.*, the basement floor, where the crushers and a portion of the conveying machinery are installed; the pump floor, containing the heater tanks and pumps; and the mixing floor, where the mixing tanks and daily supply hoppers with all their appurtenances are located. The motive power for crushers, conveyors, elevators, mixers, etc., will be electricity on a 500-volt circuit. All motors will be of the latest type, having commutating poles, and will be operated from one switch-board.

The daily supply hoppers will have a capacity of 900 cu. ft., and will be supported on four pairs of helical springs, as shown on Plate

XVI. Each spring will carry a safe load of 10 000 lb., and deflect ½ in. under a load of 1 250 lb. The purpose of having this hopper move vertically is to give automatic control of the conveyors feeding the hoppers. When the hopper is full, it will stand at its lowest elevation, but as the contents are withdrawn the hopper will rise, and, on reaching a certain point, will automatically switch in the current driving the motors which operate the feeding conveyors. As the hopper becomes filled again, it descends and again cuts off the current, stopping the feeding conveyors.

The lime is fed from its daily supply hopper into automatic scales, each of which dumps the required amount into its tank, at regular intervals, varying from 1 to 4 minutes. The scales are designed to weigh any amount from 30 to 120 lb. at a single load, the frequency of dumping also being adjustable between intervals of 1 and 4 min., the two adjustments giving quantities varying at the rate of from 10 to 80 lb. per minute.

The sulphate of iron is delivered from its daily supply hopper into the solution tanks by the feeders previously described.

The milk of lime flows from the mixing tanks into one of the heating tanks on the pump-room floor and thence to the collecting tank from which it is taken by the pumps. The collecting tank is supplied with a varying quantity of cold water, controlled by a float valve, for the purpose of cooling and diluting the milk of lime, and also for increasing the volume of dilute milk of lime, so that the pumps shall not empty the tank at any time because of the varying quantity of milk of lime flowing from the mixing tanks, or because of the variable speed of the pumps themselves on account of variation in voltage of the current supplied.

The conveyors and elevators are shown in plan and elevation on Plates XVII and XVIII, and Fig. 3. The lump lime is unloaded from the car into the crushers, A and B, on the east side of the building. It is crushed to ½ in. and smaller, and carried by Conveyors A and B to Conveyor C, which dumps it into the boot of Elevator D, by which it is elevated to Conveyor E, which runs above the lime storage bins and dumps into these bins through doors provided for that purpose.

In taking lime from the bins for daily use, the crushed lime feeds by gravity into Conveyor F, which delivers it to Elevator G, which,

FIG. 1.—INTERIOR OF COAGULANT HOUSE, JUNE 13TH, 1905.

FIG. 2.—INTERIOR OF COAGULANT HOUSE, APRIL 9TH 1907.

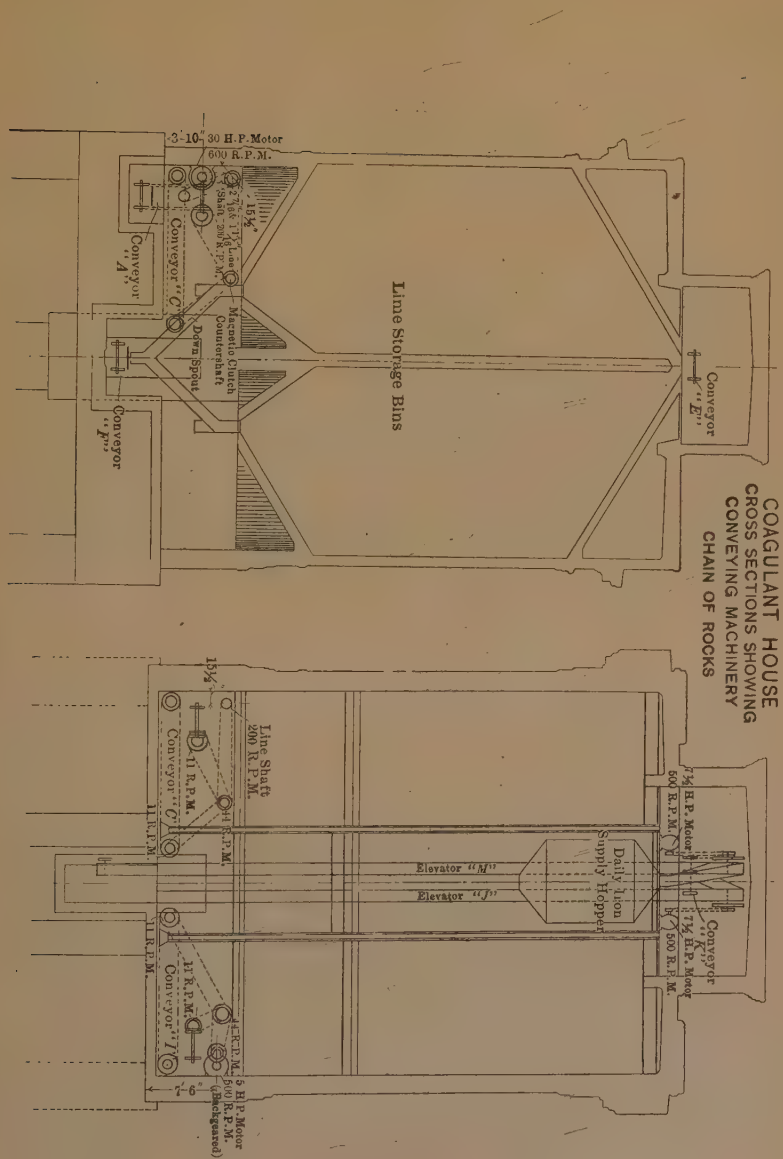

COAGULANT HOUSE
CROSS SECTIONS SHOWING
CONVEYING MACHINERY
CHAIN OF ROCKS
Lime Storage Bins
Conveyor "E"
Conveyor "A"
Conveyor "C"
Conveyor "F"
Magnetic Clutch Countershaft
Down Spout
30 H.P. Motor
600 R.P.M.
Line Shaft 200 R.P.M.
Elevator "H"
Elevator "J"
Daily Iron Supply Hopper
Conveyor "K"
Conveyor "I"
5 H.P. Motor
500 R.P.M.
(Backgeared)

FIG. 3.

in turn, carries it up to the lantern and delivers it to the daily supply hopper.

The sulphate of iron will be unloaded from cars on the west side of the building by an automatic power shovel into Conveyor H, whence, by Conveyors H and I, Elevator J and Conveyor K, it will be delivered into the storage bins. It will be taken from the bins by the same method as the crushed lime, by Conveyor L and Elevator M, to the daily supply hopper. As before mentioned, the motors driving the conveyors and elevators feeding the daily supply hoppers will be started and stopped automatically by the vertical motion of the hoppers. The daily supply hoppers will each carry from 50 000 to 60 000 lb. of material, which is sufficient for from 2 to 2½ days' supply of sulphate of iron, and from 16 to 20 hours' supply of lime, at the present rate of use.

In case any of the machinery feeding the daily supply hoppers breaks down, provision is made for drawing either sulphate of iron or lime, or both, from the bottoms of the bins into small cars on the basement floor, from which they will be elevated by the freight elevator, shown on Plate XIX, to the mixing floor, when the tanks will be fed by hand, as is being done at the present time.

The cost of operating the new plant is estimated at $17 000 per year, including interest and depreciation, as compared to $21 500, which has been the average cost of operation for the past three years. The actual yearly cost of operation alone of the new plant will be at least $9 000 less than that of the original plant.

There are now being built two new settling basins, Nos. 7 and 8, having a capacity of 75 000 000 gal., at a cost of about $500 000. These basins are constructed with reinforced concrete walls, of sections shown on Plate XX. The walls are backed with clay puddle 18 in. thick. The floor is covered with 9 in. of concrete, in blocks 8 ft. square, on 18 in. of clay puddle, the joints between the blocks being filled with an asphalt filler.

These basins are located as shown on Plate XXI, and are connected to the drawing conduit, the by-pass, and Basin No. 6, in such manner that the flow of water may be changed according to any desired direction, thus making the operation of the settling basins perfectly flexible and adaptable to any conditions which may arise. Basin No. 8 is connected with the filling chamber by a 7-ft. steel-pipe connection, so that the raw water may be introduced directly into it, mak-

FIG. 1.—PRESENT COAGULANT HOUSE.

FIG. 2.—NEW COAGULANT PLANT.

ing it a primary settling basin if desired, and when Basin No. 6 is made the primary settling basin, clear water may be drawn from Basin No. 1 through the 7-ft. pipe into Basin No. 8, thus making Basin No. 7 the final basin from which the treated water is drawn. With these basins in service and the improved facilities for uniform treatment afforded by the new coagulating plant, there is no reason to doubt that St. Louis will be supplied with water as agreeable to the eye and as pure and wholesome as is enjoyed by any city in the United States.

From the inception of the first idea of the St. Louis process of clarifying and purifying the water supply, the work has been prosecuted with all the energy and skill of the Department, individually and collectively. No one man, apart from the Water Commissioner himself, who, of course, has borne the greatest portion of the responsibility of success or failure, can be considered to be entitled to more credit for its success than any other employee engaged upon that particular work. It would not be just to conclude this paper without mentioning the valuable and efficient services of Mr. Arthur I. Jacobs, Engineer in Charge of the Supply and Purifying Division, and Mr. Wilson D. Monfort, Chemist of the Department, both of whom have contributed largely to the success of the process.

DISCUSSION.

Mr. Burgess. PHILIP BURGESS, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has been engaged for the last year and a half in making tests of the various water purification plants in Ohio, for the State Board of Health, and has read with much interest Mr. Wall's paper on the design and operation of the St. Louis plant. There are, however, two or three points in the operation of this plant which differ from the method ordinarily used at filter plants, and are not discussed in the paper. Among these is the practice of using such large amounts of lime, with a consequent reduction in the hardness of the treated water.

It is stated in the paper that the amount of lime used is as much as the water will stand, and that lime is added until the water in the last basin is slightly caustic. This means that, while the river water contains generally two or three parts of carbon dioxide per million, and hence no normal carbonates, all the free and half-bound carbon dioxide is eliminated by the treatment, producing a water that contains only normal carbonates and, at times, even a slight caustic alkalinity. The question arises: is the use of a large amount of lime necessary for proper coagulation? The chemical reactions, as stated in the paper, show a reduction of the bicarbonates by the treatment, and the use of lime in amounts beyond that necessary for the removal of the acid iron salts formed by addition of the copperas solution to the raw water.

The following statement gives the results of the reactions of the coagulants in those filter plants in Ohio where copperas and lime are used as coagulants. There is first a reaction of the copperas on the raw water, before lime is added, causing a reduction of the bi-carbonate alkalinity and the formation of acid iron salts. Lime is then added to neutralize the iron salts, and the total alkalinity is brought back to about the original content of the raw water. A further increase in the amount of lime added gives an increase in the amount of total alkalinity, a condition which obtains until a point is reached at which a softening action by the lime begins. Beyond this point the further addition of lime results in the decrease of the total alkalinity, a reduction varying with the amount of lime used, and similar to that at all water-softening plants. The amount of normal carbonate present before there is any softening action has, in the winter, been found to be as high as 32 parts per million, even 6 hours after the coagulants had been added.

In many filter plants where lime and copperas are used as coagulants, it is the custom to add first the copperas and then the lime until a sufficient quantity of the latter is present to give a treated water which is slightly alkaline to phenolphthalein. In general, it may be said that the use of 1.0 gr. of copperas per gallon requires 0.4 gr. of

lime per gallon (in bulk) to give a water containing a small amount of normal carbonates. Whether or not there is a final reduction of the total alkalinity of the treated water, seems to depend on the character of the raw water, its temperature, and somewhat on the time which elapses between the introduction of the coagulants. Mr. Burgess.

At Marietta, Ohio, there is a filter plant using lime and iron for coagulants, where very careful records of the operation of the filter plant are kept; the results there are characteristic of those at similar plants where lime and iron are used. Table 7 shows the monthly averages of the daily determinations made during the last 6 months by Mr. George M. Gadsby, who has been retained as chemist at the plant.

The most striking feature of Table 7 is the great variations in the bacterial efficiencies obtained by the basins, namely, from 50 to 94.4 per cent. The operation of the plant was somewhat hindered during March by the very high stage of water in the Ohio River, which resulted in a decrease in the efficiency of the coagulating basins, but the chief cause of the poor results obtained during March and April was that not quite sufficient lime was used. The necessity of having the water slightly alkaline to phenolphthalein was not appreciated, and the effluent from the filters was frequently slightly acid to phenolphthalein. It should be noted, however, that practically no iron went through the filters, even under this condition. Beginning on May 1st, more attention was paid to having a small amount of normal carbonates in the treated water, with the result that the efficiency of the basins increased immediately to nearly 90 per cent. In other words, the use of a very few more pounds of lime increased the efficiency 20 per cent.

The tendency to use the least possible amount of lime was not due to inadvertence, but to the fact that previously, when lime was used in such quantities as to give 20 to 40 parts of normal carbonates per million in the treated water, the air pipes in the filters were clogged by lime incrustants and had to be removed and cleaned. It was with the intention of avoiding a recurrence of this condition that the least possible amount of lime was used.

At Lorain, Ohio, lime and copperas have been used as coagulants in the old purification plant since 1903. Lime is generally used in such quantities as will give a small amount of normal carbonate in the effluent, but the occasional use of too large amounts of lime, where the coagulation period is only 20 min., has caused the formation of after-deposits of lime on the grains of the sand, resulting in an increase in the effective size from 0.52 to 0.61 mm. By analysis, the sand in the filters was found to be covered with lime to the extent of 37% of its weight. It is manifestly impossible to obtain satisfactory results from the filters under such conditions. It has been stated that

TABLE 7.—Analyses of Raw Water and Effluent from Filters, Marietta, Ohio.

(Parts per Million.)

	March.		April.		May.		June.		July.		August.		Average.	
	Raw.	Eff.	Raw.	Eff.	Raw.	Eff.	Raw.	Eff.	Raw.	Eff.	Raw.	Eff.	Raw.	Eff.
Turbidity	300	0	175	0	220	0	170	0	1 500	0	250	0	435	0
Color	6	8	6	6	10	6	13	15	18	15	10	8	10.5	9.7
Total alkalinity	17	21	18	20	22	27	25	30	30	33	28	31	23	27
Phenol alkalinity	0	0	0	0	0	+2	0	+3	0	+2.5	0	+3*	0	+1
Incrustants	33	42	35	46	21	34	19	32	20	42	43	55	28	42
Total hardness	50	63	53	66	43	61	44	62	50	75	71	86	52	69
Calcium			18	23	15	20	15	24	21	31	27	34	19	26
Iron			0.05	0.08	0.11	0.06	0.11	0.11	0.10	0.09	0.06	0.05	0.09	0.08
Copperas, in grains per gallon	3.38		1.17		2.06		2.14		3.25		1.65		2.27	
Lime, in grains per gallon	0.95		0.82		0.83		0.85		1.23		0.67		0.89	
Bacterial efficiency of coagulating basins	50.0%		67.4%		87.8%		80.6%		94.4%		84.8%		77.5%	

* Estimated.

Raw water contains from 2 to 4 parts per million of free carbon dioxide. Length of period of sedimentation = 3.5 hours.

Mr. Burgess.

a rapid application of wash-water tends to remove these after-deposits of lime, but the writer has no definite information on this point. Mr. Burgess.

From the foregoing facts, it is apparent that the proper use of lime and copperas, as coagulants for filtration, requires a constant and accurate adjustment between these two materials, such that the treated water shall contain a small, and only a small, amount of normal carbonates. Under such conditions, there is caused an increase in the total hardness of the water, amounting to about ½ gr. per gal. for each grain of copperas added, accompanied by slight change in the temporary hardness and a removal of about 33% of the color. It will appear, also, that in the lime and copperas treatment, as applied at St. Louis, not only is the coagulation obtained from the copperas, but the treatment is one of softening the water. The coagulating masses formed by the softening treatment are large, flocculent, and very efficient in the removal of suspended matters and bacteria; and to this is probably due much of the success of the plant at St. Louis. Also, the large removal of color at St. Louis is undoubtedly due to the softening treatment, without which probably only one-third of the dissolved color would be eliminated.

The use of such large quantities of lime, where sedimentation and clarification alone are relied on for the purification of the supply, is shown to be necessary at such plants as that at St. Louis; but, from the foregoing discussion, it would seem to be shown that this practice is not advisable in connection with the coagulation of water applied to filter plants, on account of the clogging of the sand and pipes by after-deposits.

Edward Prince, M. Am. Soc. C. E. (by letter).—No one can read this interesting and instructive paper without a feeling of satisfaction at the recital of efforts ending in success so well earned. Mr. Prince.

In 1846 the writer first knew St. Louis water, when, on his way to college, he visited his brother, David Prince, M.D., Professor in Pope's College, whose office was at the corner of Fourth and Olive Streets (then the heart of the city). It was the habit of Dr. Prince to draw two buckets of muddy water from his faucet, at about 6 p. m., and set them away to settle over night, say 12 hours. The next day he would use the rather milky-looking water for drinking and lavation. This was the general custom, and few used a strainer of any kind, although thousands of acres of excellent tripoli existed in untold quantities in Missouri. Some few people—laundrymen, and others—used a little alum to clear the water over night, for rinsing clothes on the following day. About ⅜ in. of jelly-like mud was deposited at the bottom of the clear-water buckets after 12 hours' subsidence. The popular opinion then was that the Missouri mud was very healthful, and promoted good digestion.

The St. Louis water also possessed another excellent quality, as

Mr. Prince. proven by the fact that the United States Navy sent to St. Louis for quantities of Missouri River water for use on long cruises, and particularly for use on the voyages in the South Atlantic, because this water never soured nor turned to bilge-water aboard ship, as most waters would.

When the writer was building the water-works at Quincy, Ill., in 1873, and operating them in subsequent years, he became very well acquainted with the late Thomas J. Whitman, M. Am. Soc. C. E., then Chief Engineer of the St. Louis Water-works, and at one time an honored Vice-President of this Society.

Several times Mr. Whitman showed the writer the subsiding reservoirs in operation. At that time there were four of these basins, each of 16 000 000 gal. capacity, or about a day's supply in all.

The quantity of silt deposited seemed to be enormous, and, of course, this deposit had to be removed. These subsiding reservoirs, as remembered by the writer, were constructed of cut stone, with vertical sides on the water faces, and as shown in Plate XXI, and in each, at 1, 2, 3, and 4, as shown by a double line, there was a sluice, about 5 ft. wide and 1 ft. deep, having a gentle slope, and communicating with a sewer running to the river. The outside of the bottom of each reservoir was a little higher than the sluice at any point, and the slope was probably about 1 in 100.

To get rid of the deposit was a serious problem. Mr. Whitman tried all probable ways, such as the use of hose nozzles, solid and spray, large and small, with powerful pumps, but in vain. The jets simply honeycombed the, say, 10 or 12 in. of thick, caked, jelly-like mud.

He finally adopted the following simple method: The reservoir to be cleaned was drained, and, commencing at the upper end, a sufficient number of men were provided with stout-handled push-boards, say 10 or 12 in. wide, and 2 or 3 ft. long, probably with one light steel edge for cutting, and it was really surprising to see these men move to the sluice so easily such large cakes of silt. Then, the water let in at the sides, for the purpose of lubrication only, completed the transportation to the river. Considering the condition of the silt, after several weeks of settlement and compacting, this, no doubt, was the best way to clean out the reservoir and get rid of the deposit at the least cost.

It is interesting and remarkable that, at the intake, Chain of Rocks, St. Louis gets its water mostly from the Missouri River, although the two rivers unite several miles above this point. After the Missouri River (often called The Big Muddy) empties into the Mississippi, the waters do not commingle, and, usually, the line of demarcation can be seen, by the difference in the color (turbidity) of the two rivers, for miles below the city. This and many of the fore-

going remarks have been made simply to emphasize the fact that it was a great problem at St. Louis to obtain clear water economically, and in great quantity, from water holding so much solid matter in suspension. Mr. Prince.

Some credit, also, is due to Mr. Charles R. Henderson, former Superintendent of the Quincy Water-works, for first having found out the proper proportions of chemicals to use in the process of water coagulation and subsidence resulting in clear water. He is now connected with the water-works at Waterloo, Iowa, and has exerted himself very cheerfully to impart all the information gleaned from very long continued and carefully made experiments.

The methods and appliances in use and to be brought into use in St. Louis for the purification of the water are the results of persistent study and experiment. They are quite an advance on the Quincy methods, in many respects, and seem to be very complete.

It is possible, however, that some cheaper coagulant may be discovered, which will dispense with the use of sulphate of iron; and yet, at the present time, sulphate of iron as a coagulant and lime (water or milk) as a precipitant have no successful rivals for water-works purposes.

It is likely that the addition of subsiding reservoirs Nos. 7 and 8 will be found to be worth very much more than they will cost, for obvious reasons. With the evidently rapid growth of St. Louis, it is likely that still more subsiding reservoirs, and possibly the enlargement of the intake at Chain of Rocks, will become necessary and desirable.

George A. Soper, M. Am. Soc. C. E.—All should feel indebted to Mr. Wall for his description of the method of purifying the public water supply of St. Louis, and particularly for his description of the method of handling the large quantities of chemicals required. Mr. Soper.

As to the effects of the process, it seems to be too soon to form a final opinion. There have been abundant illustrations elsewhere of the usefulness of chemical precipitation, and of precipitation with iron and lime, but chemicals are generally used as a preliminary to filtration.

It will necessarily be some time before the sanitary value of the St. Louis works can be accurately measured. There is no doubt that the water has been improved, and improved materially, and apparently at a comparatively small cost; but the measure of that improvement depends on facts and figures which are not yet available. The speaker would like to see more analytical and statistical data concerning the effects of operation.

There are, of course, drawbacks to the use of chemicals, in treating a water supply. All these drawbacks are not self-evident. One objection results from the changes sometimes produced in the mineral

Mr. Soper. content of the water. The speaker does not know that there has been any trouble with boilers or distributing pipes in St. Louis, but it is one of the things to look out for.

Considerable interest attaches to the author's remarks concerning the diminution in typhoid fever which seemed to follow the introduction of the plant. The use of typhoid statistics, in measuring the effect of an improved water supply, has many pitfalls and difficulties, and it would not be strange if the author had fallen into one or more of them. It is, unfortunately, unsafe to rely even on the vital statistics collected in many of the most advanced and best governed American cities, for, in the first place, physicians do not always know typhoid fever when they see it, and, in the second place, they do not always report it when they recognize it.

The speaker would like to know, if convenient to the author, how many cases of other fevers, which might have been typhoid, occurred during the period covered by his typhoid statistics. The statistics seem to be in error, because it appears that there has been a case mortality of from 14 to 25%, while it is probable that not more than 8% of all people attacked by typhoid die of it.

Even if the few statistics given were as accurate as could be desired, they would not of necessity show much of importance concerning the operation of the works. It is not easy to understand how much typhoid, in a city like St. Louis, is due to water. There are usually many other sources of this disease. It seems to be quite possible that, if all the facts were known, it would appear that the water supply of St. Louis has had in the last few years very little to do with the prevalence of typhoid, and that the disease has not increased or diminished to the extent commonly supposed.

There is a final point, which the speaker hesitates to mention, but inasmuch as, before this Society, it will probably be taken in the conservative spirit in which it is intended, and may lead to useful inquiries, it may be referred to briefly. It concerns the composition of the sulphate of iron used. How much iron is present, and how much acid? What is the quality of the lime? The chemicals cannot be pure. No city could afford to pay for pure chemicals, even if they were obtainable. What are the impurities? How much arsenic is there in this sulphate? Some years ago the speaker had occasion to examine specimens of sulphate of aluminum from a good many filter plants, and found arsenic in nearly all of them. It is true that, usually, the arsenic was not present in large quantity, but it was easily discoverable, and in some of the samples it was present in sufficient amount to be of more than passing interest. The arsenic, of course, came from the sulphuric acid used in making the sulphate of aluminum, the sulphuric acid having been produced from pyrites which contained arsenic.

Mr. Whipple.

G. C. WHIPPLE, ASSOC. M. AM. SOC. C. E.—There are a number of questions that the speaker would like to ask in regard to this interesting paper, especially as to the details of the method by which the chemicals are applied to the water. He is more interested, however, in the general principles of the process and in the results, than in the details. This method of purification by chemical precipitation is usually regarded as a preliminary process, to be followed by filtration; but in this case the preliminary process is stretched so as to perform all the work that is done. The results which have been accomplished by this stretching of a preliminary process are of great interest, and no doubt the people of St. Louis appreciate very much the improvement that has been made in the quality of their water supply.

It seems to the speaker, however, that the author takes altogether too roseate a view of the future. For instance, near the close of the paper is found this statement:

"With these basins in service and the improved facilities for uniform treatment afforded by the new coagulating plant, there is no reason to doubt that St. Louis will be supplied with water as agreeable to the eye and as pure and wholesome as is enjoyed by any city in the United States."

This is certainly an exaggeration. The speaker cannot believe that the results of this process are going to be as good as the results which would be obtained by supplementing chemical coagulation with filtration, according to modern methods; and it does not seem to him that the figures presented in the tables bear out the statement just quoted. In the early part of the paper the author speaks of the tap water in St. Louis in 1904 as being clear and sparkling. To translate this into the language used by the chemist would be to say that the water had a turbidity of zero. The tables in the paper show that the turbidity of the water has not always been zero, but often far from it; or rather, they show that the suspended matter has not been zero, for it will be noticed that figures for the turbidity of the tap water are not given at all. They are simply given for the water before treatment. The speaker hopes that, in his closing discussion, the author will supply this deficiency.

Of the few analyses given, some are monthly averages, and others were taken a week apart. Now, it is a well-known fact that a monthly average often does not tell what the condition of the water has been from day to day. The figures for a month may be fairly low and satisfactory, and yet there may have been days during that month when the water was not satisfactory. This the speaker knows from experience, and there is no reason to believe that it is not true of the St. Louis water. That the product of such a plant as this would be irregular, would naturally be expected, for while the application of the chemicals may be under control, the natural physical and meteorological conditions which affect the sedimentation are not subject to control.

Mr. Whipple. The speaker had the pleasure of visiting the St. Louis plant, during the early part of the work, at a time when it was not working as well as it evidently has been lately, but he then saw an illustration of its irregularity. For the month of June, 1904, the amount of suspended matter in the effluent is given in Table 1 as 39 parts per million, and yet on the day when the speaker was there the turbidity of the water leaving the settling basins was 400. Therefore, it seems to the speaker that the figures given may be, to a certain extent, a little misleading, and may give an idea that the water is clearer than it sometimes really is. Countenance is lent to this view by reports which have come from persons who have visited the city and who, while they remark that the quality of the water has been very greatly improved, speak of its occasional turbidity.

The speaker agrees with Dr. Soper that the data are hardly sufficient to give the water supply an absolutely clean bill of health, and that they do not show what may be the final results of this treatment, from a sanitary point of view. The figures, admitting their accuracy, do show a great reduction in the typhoid fever death-rate, but if one were to follow the typhoid fever records back for a number of years before those tabulated, he would find that about ten years ago the typhoid fever death-rates were nearly as low as they have been during the last year or two, and that this was during the time when the city was depending on plain sedimentation, without the use of chemicals, for the purification of the supply.

Here it should be noticed that the author refers to the efficiency of the old sedimentation process as being "from 10 to 80 per cent." Now, the report of the commission which studied the water supply in 1904 gave the efficiency of the settling basins as "from 75 to 94%," and gave the average efficiency as 85%, while, if the speaker recollects correctly, later observations made by Mr. Flad showed the efficiency of the sedimentation process without chemicals to be more than 90%, a figure which comes closer to the figures given for the chemical process in the paper.

The speaker mentions these matters because it seems to him that this paper might be made of very much greater value, and certainly much more convincing, by giving more detailed information regarding the quality of the treated water. Some of the omissions were doubtless accidental, for the author speaks in one place of tables giving the results of turbidity, and in another place of tables giving the results of tests for *B. Coli* and other tests, which data are not found in the places mentioned.

One other point might be mentioned, namely, the effect which the use of these chemicals may have on the mineral quality of the water. It is well known that, where lime is used, the reaction is not instantaneous, and there are apt to occur what are known as "after

deposits" of calcium carbonate, etc. These deposits may take place after the water has left the settling basins. The speaker once visited a city where there were considerable deposits of lime on the inner surfaces of the service pipes and on the working parts of meters. These deposits were not very large, but were quite noticeable. There is no reason to doubt that in time this will happen in St. Louis. Mr. Whipple.

The question of the quality of the ferrous sulphate used has been raised by Dr. Soper. This is an important matter, for the chemical may conceivably contain substances which may be detrimental to the pumps through which the water passes.

In conclusion, the speaker wishes to congratulate the City of St. Louis for what its engineers have done. They deserve praise for the energetic manner in which they have taken hold of the problem. He hopes, however, that the city will not content itself with a half-way process, but will use as much energy in securing a complete process of purification as it has used in developing this preliminary process of chemical precipitation.

L. L. Tribus, M. Am. Soc. C. E.—The speaker has read this paper with considerable interest, and desires to call attention to one thing in particular—and a very commendable matter, at that. While St. Louis was discussing, and had been considering for years, the question of an improved water supply, with experts from all over the country, Boards of Trade, Chambers of Commerce, citizens, etc., yet no one seemed to be able to suggest any really practical method of doing the work, until the gentlemen mentioned in this paper, having to meet the situation, designed a plant which has done the work, and has given water of vastly better quality than that with which the city had ever been supplied. That is a true engineering feat, and deserves high praise. Mr. Tribus.

It is very probable—and the speaker does not disagree with either Mr. Soper or Mr. Whipple—that better results can be secured by carrying the process forward, by filtering the water after it passes through the sedimentation tanks, or by greater chemical refinement; but the main point stands out that the engineers met a difficult situation, and built a practicable plant in a short time, and at a comparatively small cost.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—This problem has engaged the closest attention of engineers for many years, and the general conclusion has been that sedimentation—followed by coagulation and filtration—was the proper solution. The expense of such purification approximates from $15 to $17 per million gallons. Mr. Le Conte.

The author's recent experience at St. Louis, however, seems to show that, for all practical purposes, the whole operation can be performed successfully by coagulation, followed by prolonged sedimenta-

Mr. Le Conte. tion, thus apparently shutting out entirely the expensive necessity for filtration. According to his experience, the water can be purified for $5 per million gallons. Of course, all have understood, heretofore, that it is physically possible to accomplish the entire operation of clarification by coagulation and subsidence; but it has always been maintained, with much force and judgment, that it was unwise to do so, on a large scale, because a considerable portion of the coarser sediment could first be removed more cheaply and successfully by plain subsidence, and, most important of all, that the last traces of clay particles could be more completely removed by filtration than by a prolonged settling.

Furthermore, engineers have generally held that good reliable sanitary requirements, as to bacterial efficiency, could best be accomplished and properly maintained by filtration. The author now states that the same bacterial efficiency can be had by simple coagulation and prolonged subsidence, and that the result is accomplished at one-third the cost. This is a most important fact. It would appear, however, by reference to the various tables submitted, that, in point of fact, the clarification is not by any means such as would be desired by a fastidious community. The average "color" is given as 10 to 12, whereas the effluent from good filters generally shows 0 to 5. That is to say, it is more than four times as cloudy as good filtered water.

Table 4 shows that in November, 1906, and in January and February, 1907, the bacterial efficiency was below standard, and that in March, 1907, for the first time in three years' operations, it was above standard. That is to say, from a sanitary point of view, the water delivered to consumers was unsafe most of the time except during the last month of operations.

It would be interesting to know whether or not the maximum quantity of sulphate of iron (3.75 gr. per gal.) and the maximum quantity of lime (11.0 gr. per gal.) used during the third year of operation were actually consumed during March, 1907, when the bacterial efficiency was satisfactory.

The writer is greatly impressed by the good results in Table 2, showing the reduction in the typhoid death-rate, due, undoubtedly, to water purification. This feature, above all others, appeals to one's sense of humanity. The results stand out in strong contrast with the late experience in Washington, D. C., where, apparently, the filters failed to lower the typhoid death-rate, as was reasonably expected.

There is no doubt, however, that the bacterial efficiency attained in the St. Louis experiments is practically all that could be desired, from a sanitary point of view, and the expense incurred, namely, $4.62 per million gallons, is only about one-third of that of good filtered Mississippi River water.

The whole question, then, turns upon the degree of clarification

necessary or desirable in each case. Each city is now at liberty to decide for itself according to its financial ability, and later, perhaps, when better able to stand the extra burden, may introduce filtration. This will prove to be particularly good news to many western cities now supplied with muddy river waters. Mr. Le Conte.

EDWARD E. WALL, M. AM. SOC. C. E. (by letter).—The writer regrets that, in his paper, the question of the determination of the proper amounts of lime and sulphate of iron to be added to the river water was passed over in such a cursory manner as to leave the impression that a necessity of the process was caustic alkalinity in the basins. When the process was first inaugurated, and for a considerable time afterward, this was thought to be true for all conditions of the river water, and, in the struggle to maintain its clarity, the factor of caustic alkalinity in the treated water did not receive the attention it deserved until almost a year later, and it was not until the spring of 1906 that the problem of proportioning the lime and sulphate of iron was made to depend on other variables than the appearance of the water in the basins, supplemented by a few crude tests. Still, with all the assistance afforded by frequent determinations of alkalinity and turbidity, the increase and decrease of the quantities of chemicals added must be influenced somewhat by the appearance of the water in the basins. For example, it may happen that the alkalinity conditions in the basins are satisfactory and that the turbidity readings are normal, but an inspection of the basins may show that the coagulation is finely divided and carrying over into the third or fourth basin. This condition would usually be met by an increase in the sulphate of iron, followed later by an increase in the lime if necessary. On the other hand, should the coagulation show in large flocculent masses, settling out very rapidly and close to the inlets, the quantities of lime and sulphate of iron would be reduced, regardless of the low caustic alkalinity in the basins. Mr. Wall.

When the river water is carrying a large quantity of suspended matter, the changes in the amounts of lime added follow approximately the fluctuations of alkalinity in the river water; but, when the amount of suspended matter becomes low for the Mississippi River, this tendency is not as noticeable. The changes in the quantities of sulphate of iron vary after a rough fashion with the amounts of suspended solids from day to day. This is shown on Plates XXVII and XXVIII.

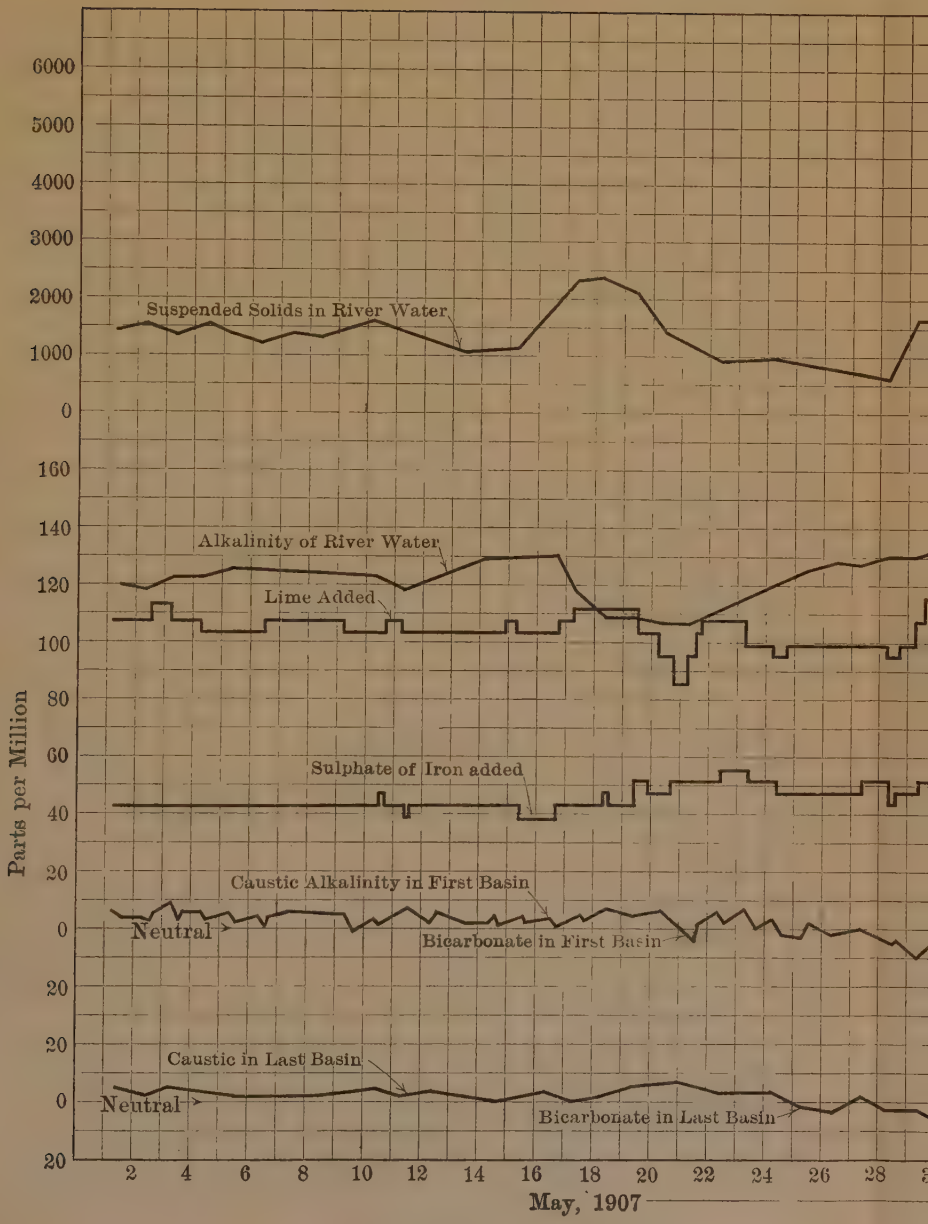
The two lower profiles on each of these plates show the variations from caustic alkalinity to bicarbonate in the first or entrance basin, and in the last one of the series. An inspection of these plates will show how the fluctuations afford a key to the changes necessary to be made in the quantity of lime added. As the caustic alkalinity in the first basin rises, the lime is reduced, and *vice versa*. The caustic

Mr. Wall. alkalinity and the bicarbonates in the last basin follow approximately the changes in the first basin, although they are not as pronounced, and usually occur a little later.

Plate XXVII shows the conditions during May and June, 1907, when the river water was carrying a large quantity of suspended solids.

Plate XXVIII shows the changes during August and September, 1907, when the suspended solids dropped from 2 500 to 500 parts per million, the alkalinity rising from 120 to 160 parts per million. It will be noticed that the decrease in the quantities added, both of lime and of sulphate of iron, is coincident with the drop in suspended solids; also, that the caustic alkalinity in the first basin disappears entirely during the latter part of September, although the alkalinity of the river water has increased. During November and December, 1907, and January, 1908, there has been no caustic alkalinity in any of the basins, and the bicarbonates have run as high as 40 parts per million in the first and 44 parts per million in the last basin. The alkalinity of the river water has increased from 170 to 217 parts per million during these months, the corresponding alkalinity of the treated water rising from 50 to 80 parts per million.

The writer entirely agrees with Mr. Burgess that the use of a large amount of lime, as practiced at St. Louis, would be impracticable in conjunction with filters, on account of the clogging of the sand and pipes. This difficulty was observed shortly after the process was started in St. Louis, when, at first, both the lime and sulphate of iron were added to the river water before its passage through the pumps. In a short time it was found that the screens, suction pipe, pump chambers, and valves were becoming coated with a deposit which rapidly increased in thickness. The lime was then added to the water in the delivery well, into which the pumps discharge, and no further trouble has been experienced in the engine-house. In the delivery well and through the filling conduit, however, a deposit several inches in thickness accumulated, and was removed after three years of service, at a trifling cost. In the drawing conduit, through which the treated water passes, a slight deposit is found, at present probably $\frac{1}{2}$ in. thick near the basins, and rapidly decreasing to a very thin coating farther down. This deposit is not believed to be increasing in thickness. It was formed during the early stages of the process, while the treated water was carrying caustic alkalinity most of the time, and when, also, a portion of the finely-divided coagulant was carried over in suspension occasionally. Scarcely any evidence of deposit is found in the pipes or pump chambers of the high-service engines, and certainly such deposit is not appreciably increasing. A careful examination of the distribution pipe, meters, and service pipes has not produced any evidence of danger to be feared from "after


Suspended Solids in River Water
Alkalinity of River Water
Lime Added
Sulphate of Iron added
Caustic Alkalinity in First Basin
Neutral
Bicarbonate in First Basin
Caustic in Last Basin
Neutral
Bicarbonate in Last Basin
Parts per Million
6000
5000
4000
3000
2000
1000
0
160
140
120
100
80
60
40
20
0
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20
0
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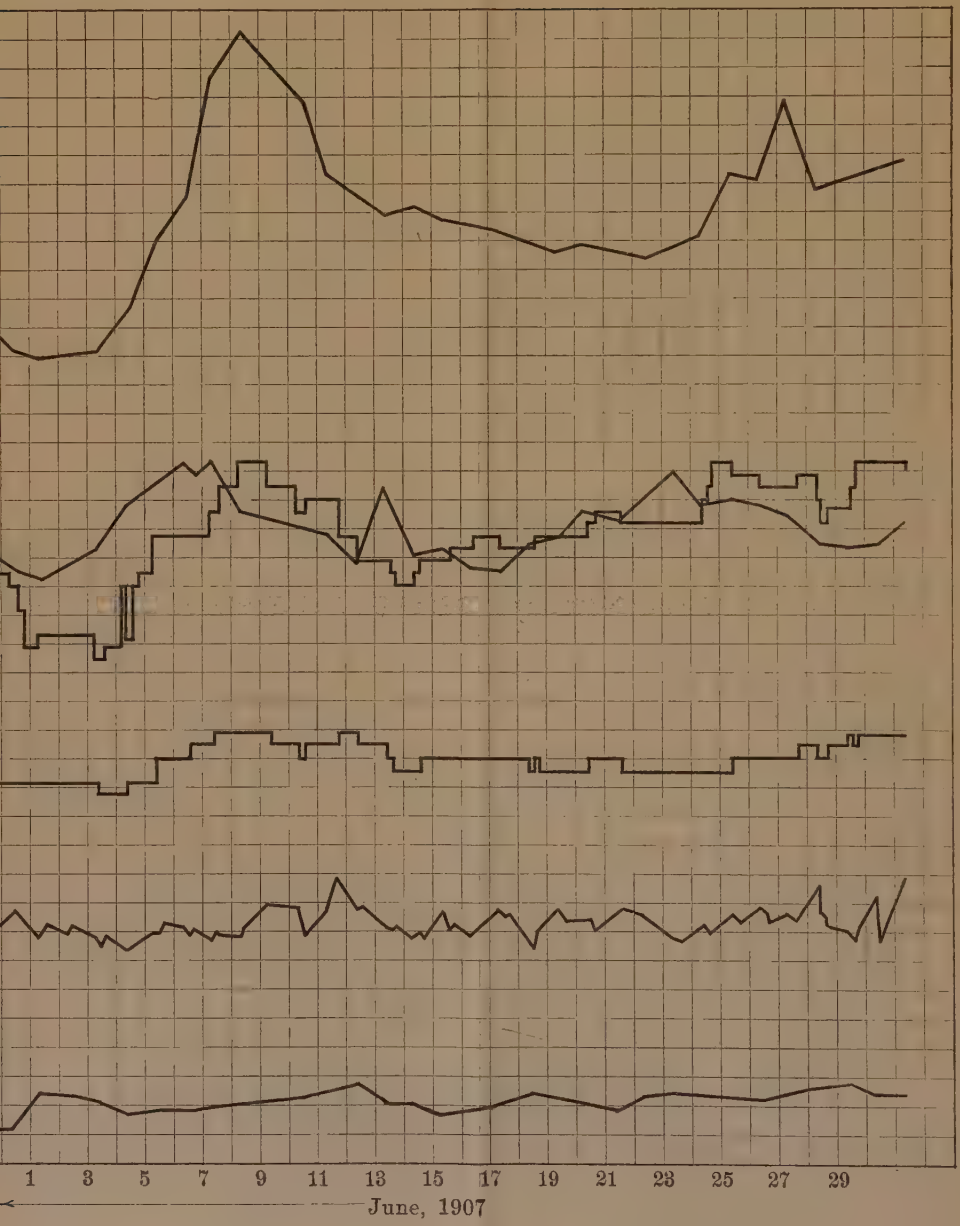
May, 1907

ECORD, CHAIN OF ROCKS, ST. LOUIS.



deposits." It is difficult to understand why any should be expected, as long as the treated water is carrying practically no suspended matter and no caustic alkalinity, which has been the case for several months, and, with additional basin capacity, it should be a matter of little difficulty to continue these conditions indefinitely. Mr. Wall.

The writer confesses that he is unable to see any basis for the opinion expressed by Dr. Soper that it is too soon to form a final opinion as to the effects of the process. It is now nearly four years since the process was inaugurated, and only beneficial effects have been observed. It would seem that, if any deleterious effect was going to occur, some indication of it should have appeared by this time. Steam users have found much less trouble in operating boilers, the scale formed being much harder, but much thinner, a better conductor of heat, necessitating less frequent cleanings and less danger of burning crown-sheets and tubes.

The writer has a copy of a report made in June, 1904, by the chemist of the American Steel and Wire Company (manufacturing the sugar sulphate), in which it is stated that no arsenic was found in any of the samples tested, and that the small quantity of arsenic in the sulphuric acid used in cleaning steel is, without doubt, removed in the cleaning tubs in the form of arsenureted hydrogen, thus leaving the liquors, from which copperas is made, free from arsenic. Even if there should be a minute quantity of arsenic in the sulphate of iron, it is scarcely possible that this could remain in the water after treatment with the quantity of lime used at St. Louis.

Sulphate of iron may contain manganese sulphate, but in such small quantities as to be negligible. The amount of free sulphuric acid in sugar sulphate is not sufficient to interfere with its value as a coagulant, the objection to its presence at all being due to the fact that it would neutralize a certain amount of lime, forming sulphate of lime, thus increasing slightly the permanent hardness of the water.

The sulphate of iron as received at St. Louis has lost a portion of its water of crystallization, so that it no longer has the theoretical composition shown by the formula, $Fe\ SO_4\ (7H_2O)$, but carries a greater percentage of $Fe\ SO_4$, thus samples are often found to contain more than 100% of $Fe\ SO_4$, 100% in this case meaning the theoretical percentage of $Fe\ SO_4$ contained in chemically pure sulphate of iron of the composition $Fe\ SO_4\ (7H_2O)$. Analyses lately made of samples taken from cars show 102.18% $Fe\ SO_4$, and, after exposure to the air for a few days, 104 per cent.

The insoluble matter in sulphate of iron is made up of sand, rolling-mill scale, and rust. These impurities are inert, as far as water purification is concerned, and amount to less than 1 per cent.

The lime usually has from 90 to 97% of CaO available, about 2% of impurities consisting of silica, magnesia, and clay, the remaining

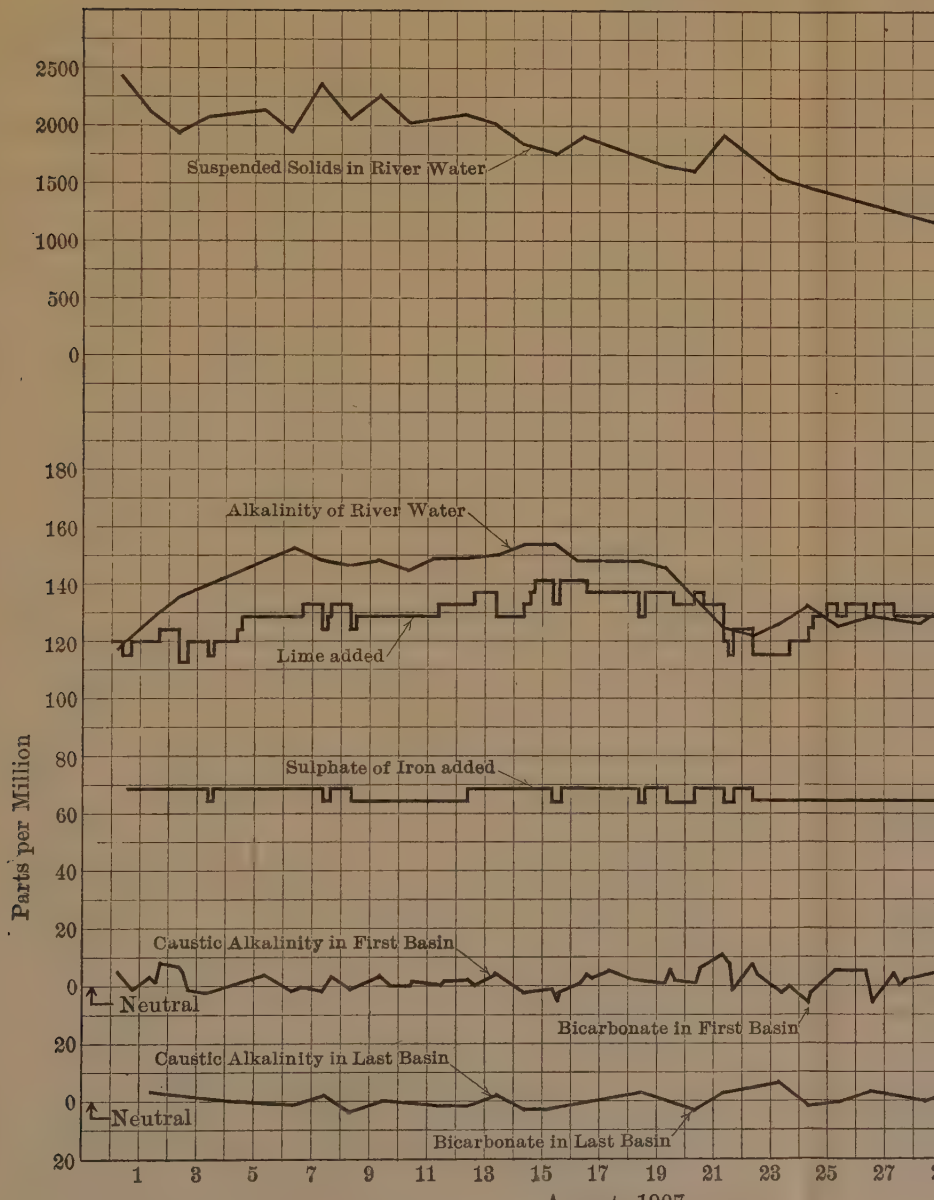
Mr. Wall. unavailable material being due to underburned stone. The lime is bought under a contract requiring it to analyze not less than 90% of CaO, all lime showing less than 90% to suffer a reduction in contract price of the same percentage that the analysis falls below 90% of CaO. Lime showing by analysis more than 95% of CaO is paid for in excess of the contract price by one-half the increase in percentage over 95% of CaO. The lime shows above 90% in CaO, except when the weather conditions are such that air slaking in transit reduces the available CaO.

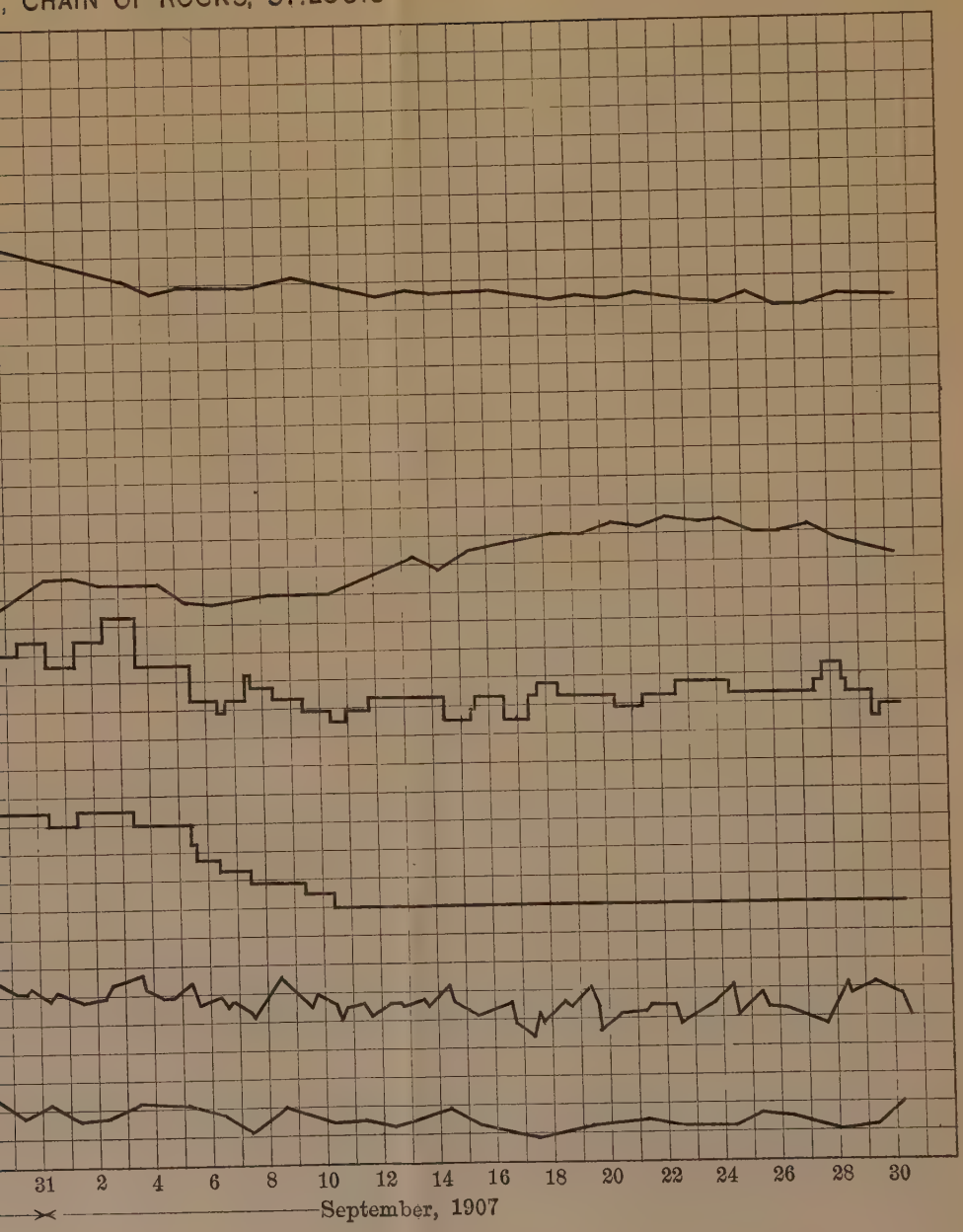
The writer believes that the effect of the purification of a water supply on the public health, as shown by the simultaneous diminution of the typhoid death-rate, is too well established to admit of any serious argument to the contrary. It may happen that an engineer, in reviewing the results of his work, is sometimes inclined to take too roseate a view of its excellence and to seize too eagerly on such statistics as at the time favor his arguments. While the writer does not wish to be understood as claiming that the entire credit for reduction in typhoid is due to the purification of the water supply at St. Louis, yet he believes that it has been the most potent factor in bringing about that result. The period of eight years, from 1900 to 1907, inclusive, which was taken in the paper as illustrative of the effect on typhoid of the water purification at St. Louis, includes the time from which the low death-rate in typhoid began to rise, due, no doubt in some degree, to the pollution of the Mississippi by the discharge from the Chicago Drainage Canal, opened early in 1900. This rise in typhoid continued until the inauguration of the purification process in 1904, when the drop was very marked, and the decrease has continued until the present time. During this period of eight years there has been no radical change in sanitary conditions in St. Louis which might affect the public health, except that of the improvement of the water supply.

Fig. 4 shows the typhoid death-rate per 100 000 in St. Louis for the last 58 years. It is interesting to compare the fluctuations on this diagram with changes in the water-works of St. Louis, and to note the remarkable regularity with which a decrease in the typhoid rate follows each improvement in the water supply.

The very pronounced drop in 1856 followed the putting into service of a new reservoir having a capacity of 45 000 000 gal., while previous to this date the settling capacity had been about 7 000 000 gal.

Unsettled conditions, brought about by the Civil War, may account for the rising typhoid rate from 1863 to 1866. In 1867 the 45 000 000-gal. reservoir was found to be almost half full of sediment, and an additional temporary reservoir was built and used in conjunction with the larger one. In 1871 the old pumping plant was abandoned and a new station was put in service at Bissell's Point, three miles farther


2500
2000
1500
1000
500
0
Suspended Solids in River Water
180
160
140
120
100
80
60
40
20
Alkalinity of River Water
Lime added
Sulphate of Iron added
Caustic Alkalinity in First Basin
0
Neutral
Bicarbonate in First Basin
20
Caustic Alkalinity in Last Basin
0
Neutral
Bicarbonate in Last Basin
20
Parts per Million
1 3 5 7 9 11 13 15 17 19 21 23 25 27
August, 1907


, CHAIN OF ROCKS, ST.LOUIS
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September, 1907

up the river, where a settling capacity of 50 000 000 gal. was provided, and the water was again pumped from these basins into the Compton Hill Reservoir, holding 60 000 000 gal., from which the distribution system was supplied by gravity. Mr. Wall.

The sudden increase of typhoid in 1892 was attributed to the contamination of the water supply by the effluent from Harlem Creek, which drained about 8 000 acres and emptied into the Mississippi just above the water-works intake. In 1893 the dry-weather flow of this creek was diverted into a channel emptying below Bissell's Point.

In 1895 the intake was moved to the Chain of Rocks, seven miles above Bissell's Point, where the settling basin capacity was increased to four times its previous quantity, in effect giving three full days' settlement to the river water before it was pumped to Compton Hill Reservoir, whereas there had been less than one day's settlement before this time. This will serve to account for the very low typhoid rate prevailing until 1900 when the Chicago Drainage Canal was opened.

When the use of typhoid statistics is confined to one city, and comparisons are drawn from yearly records, it seems that the objections to their use, raised by Dr. Soper, can hardly apply. The errors of physicians, either in not recognizing typhoid fever cases, or in failing to report them, could reasonably be supposed to be a constant factor in affecting the record, since, in a large city, the personnel of the profession and the range and methods of individual practice will vary to a comparatively small degree from year to year.

The Health Department of St. Louis has no record of cases of other fevers which might have been typhoid, for the reason that no report of cases is required, and only the deaths from malarial fevers are reported to the authorities.

The writer can readily understand why Mr. Whipple cannot believe that the results of the St. Louis process are going to be as good as would be obtained were filtration made a part of the scheme. The writer and his associates in this work did not believe that such would be the case for a long time after the process was in operation. But, as the methods were improved and the adjustments were brought under better control, the possibility of turning out as good a water as is done by filtration plants, began to be evident, and, for considerably more than a year past, the treated water has averaged as high in clarity and purity as the effluent of the ordinary filter plant. It must be remembered that the usual filter plant in daily operation does not always give either perfect or uniform results, and, thus far, has not been productive of brilliant results, when attempted on the turbid waters of the Middle West, on a large scale. There is no reason to believe that St. Louis would have fared better than some of her sister cities, had the attempt been made to filter the Mississippi water here.

Mr. Wall. There has been no attempt to keep a record of the turbidities of the treated water, the determination of the suspended solids and the color being considered sufficient. The amount of suspended solids has been, with few exceptions, practically negligible for more than a year, and the color usually below 12. Any further reduction of color has not been attempted, for the reason that the advantage of having a perfectly colorless water does not seem to be sufficient to warrant any special effort to obtain it.

Mr. Le Conte has apparently fallen into an error, confusing the color reduction with the clarification. Filtration in itself usually has little or no effect in reducing color. The average color of the treated water at St. Louis is scarcely noticeable except by direct comparison with colorless water.

Table 1 was compiled from reports of the Health Department, for the reason, as stated in the paper, that the Water Department had not the organization at that time for proper and systematic determinations of the results of the process. After April 1st, 1906, complete records of daily determinations were kept, and Tables 3 and 4 were compiled from these. The work of the Health Department was confined to samples collected once each week.

The writer made up these tables with the idea that, in abbreviated form, they would be sufficient to show the extent of the purification. To have given the daily record would neither have added to nor subtracted from the force of the figures, since the dates used are taken uniformly one week apart, and afford a thoroughly representative statement of the working of the process covering a long period of time. It is true that, during the early months of its use, there were periods, varying from a few hours to a day at a time, when the turbidity of the treated water would run from 30 to 40, and, under extreme conditions, as high as 200 or 300, on account of the improper proportions of lime and sulphate of iron added, due to the fact that the whole process was in an experimental stage and that the Department was operating in a new and untried field, with no precedents to guide it. There was also another serious obstacle to attaining uniformity of results during the first six months, namely, that the settling basins were being changed to accommodate the new system, necessitating the cutting down of available basins for settling purposes to four and even to three at times. Those periods of high turbidity in the treated water were of short duration, however, and, for the greater portion of the time, the turbidity was much lower than is indicated by the suspended solids as given in Table 1.

Another condition, which in itself has had no bearing on the purification process, yet may have served to leave a false impression on the minds of visitors to St. Louis, who afterward commented unfavorably on the water, was that of the pipes of the distribution system. In 1904

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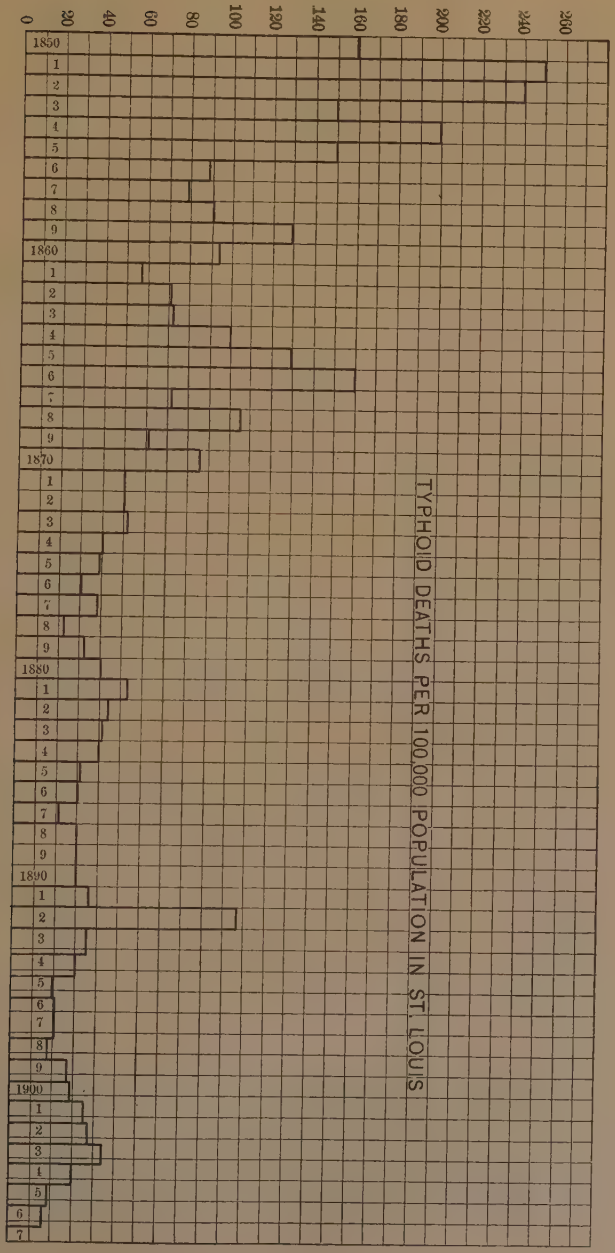
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TYPHOID DEATHS PER 100,000 POPULATION IN ST. LOUIS

FIG. 4.

Mr. Wall. St. Louis had more than 700 miles of mains, through most of which muddy water had been pumped for many years, leaving accumulations of sediment at low points, dead ends, and in lines having poor circulation. Disturbances in the direction of flow of the currents in the pipes, caused by use of fire-hydrants, or shut-offs for repairs or connections, would stir up these deposits of sediment, and affect the clearness of the water in that locality. Local complaints of muddy water were of daily occurrence for several months, but, as the distribution system became flushed out, these troubles diminished, and complaints have been rare during the past 18 months.

On looking over the records of the reduction of suspended solids by the old fill-and-draw method of simple sedimentation, the writer finds numerous cases where the reduction was less than 10%, at times when the river was low and when the suspended solids in the river water were 200 parts per million or less. The observations made by Mr. Edward Flad, and referred to by Mr. Whipple, were taken during periods of relatively high turbidity of the river water, and show an average reduction in suspended solids of 76.25% for one period of 36 consecutive days and of 67.5% for another set of continuous determinations for 33 days. The amount of suspended solids during the first period averaged 1 960 parts per million, with a minimum of 890 and a maximum of 3 267 parts per million; and, during the second period, averaged 1 305 parts per million, with a minimum of 560 and a maximum of 3 092 parts per million. Therefore the writer believes that his statement, that the method of simple sedimentation reduced the suspended matter from 10 to 80%, is fairly representative of the results of the old system.

Through an oversight, a portion of the following information in regard to tests for *B. Coli* was omitted from the paper. The writer has thought it best now to give these tests more in detail, and to add the later work done in the Department laboratory.

In June, 1905, Dr. Charles A. Snodgrass, at that time City Bacteriologist, made a series of tests for *B. Coli,* covering a period of 11 days. The results of these tests are given in Table 8.

The samples in Table 8 were taken from the river at the inlet tower, from the first basin, where the water enters, and from the last basin, where the water is drawn into the conduit on its way to the high-service pumps.

In September, 1905, an attempt was made by the Department to carry on a set of tests for *B. Coli,* but, owing to the deficiency of apparatus and lack of experience, these tests were not satisfactory. In a series of tests covering 11 days, using samples of 1 cu. cm. each day, the river water gave positive results in each case, seven of the samples of treated water gave negative results, one doubtful, and three positive.

TABLE 8.

Mr. Wall.

Character of sample.	Amount of water used.									
	10 cu. cm.		1 cu. cm.		$\frac{1}{10}$ cu. cm.		$\frac{1}{100}$ cu. cm.		$\frac{1}{1000}$ cu. cm.	
	Positive.	Negative.	Positive.	Negative.	Positive.	Negative.	Positive.	Negative.	Positive.	Negative.
Mississippi River water	.....	.....	11	0	11	0	6	5	2	9
Treated water, before sedimentation	.....	.....	8	3	6	5	1	10	0	11
Treated water, after sedimentation	1	3	0	11	2	9	0	11	0	11
Tap water, at 1624 Chestnut Street	2	3	1	10	0	11	1	10		11

Mr. F. W. Witherell, Chemical Engineer for the American Water Works and Guaranty Company, of Pittsburg, made examinations for *B. Coli* in February, 1906, the results of which are shown in Table 9.

TABLE 9.—Bacterial Examination of City Water at St. Louis, Taken from Faucet at 4219 Washington Boulevard.

Date.	Gelatine at 20° cent.		Agar.			B. Coli Communis:		B. Enteritidis Sporogenes:
			20° cent.	39° cent.				
Feb., 1906.	Water: Bacteria per cubic centimeter.	Liquefiers per cubic centimeter.	Bacteria per cubic centimeter.	Bacteria per cubic centimeter.	Acid formers per cubic centimeter.	in 1 cu. cm.	in 50 cu. cm.	in 5 cu. cm.
12.....	6 000	580	98	15	2	Negative.	Negative.	Negative.
13.....	320	12	45	14	0	"	"	"
14.....	1 040	29	4 200	310	4	"	Positive.	"
15.....	31	0	72	12	1	"	Negative.	"
16.....	20 000	89	21 000	400	0	"	"	"
17.....	270	10	4 000	35	0	"	"	"
19.....	700	31	972	29	0	"	"	"
20.....	150	21	115	21	0	"	Positive.	"
21.....	55	35	72	15	0	"	"	"
22.....	123	105	39	15	0	"	Negative.	"
23.....	45	3	50	10	0	"	"	"

In November, 1906, the Department commenced regular examinations of the river and treated water for *B. Coli.* The sterilizer did not at all times work satisfactorily, so that the results were often vitiated, and, during July, August, and September, 1907, no determinations were made. In October a new sterilizer was installed, and the work since that time has been satisfactory. In all cases the presence of *B. Coli* in the river water was established, using samples of $\frac{1}{10}$ cu. cm.

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From November, 1906, to May, 1907, samples of 1 cu. cm. of the treated water were used, with the following results:

November,	1906,	6	positive,	in	a	total	of	36 samples.
December,	"	0	"	"	"	"	12	"
January,	1907,	6	"	"	"	"	48	"
February,	"	7	"	"	"	"	38	"
March,	"	5	"	"	"	"	52	"
April,	"	11	"	"	"	"	52	"
May,	"	7	"	"	"	"	26	"

In the latter part of May, 1907, and for all *B. Coli* determinations since that time, samples of 5 cu. cm. of treated water were used, giving results as follows:

May,	1907,	6	positive,	in	a	total	of	30 samples.
June,	"	6	"	"	"	"	25	"
October,	"	3	"	"	"	"	90	"
November,	"	0	"	"	"	"	120	"
December,	"	0	"	"	"	"	90	"

The writer believes that the improvement which might be made over the present condition of the water supply of St. Louis, by supplementing the process with filtration, would be so slight that the difference could only be discovered by close analysis, and would be immaterial to the consumer. The material advantage (if any should exist) gained by filtration, would be so much out of proportion to the cost of constructing, maintaining, and operating filters, that it would be an absurdly impractical proposition to advocate, unless some new and unexpected difficulty should arise to vitiate the results now being achieved. Thus far, nothing has been discovered which would lead the most skeptical to doubt that the future of this process in St. Louis will be in the direction of uniformity of effluent, equivalent in appearance and quality to that obtained by filtration. The water delivered to consumers during the past 5 months has not differed appreciably, either from a chemical, biological, or æsthetic point of view, from filtered water, except in the fact that the St. Louis treated water has the advantage of being softened considerably by the process. This process is not applicable to all waters, by any means, because its success depends mainly on the presence of the bicarbonates of lime and magnesia in the raw water. In most cases, cities drawing their water supply from the alluvial streams of the Mississippi Valley can adopt this system with profit and satisfaction.

The writer does not believe that he is too optimistic in saying that the work done at St. Louis opens a new field for water purification which will eventually be developed into something simpler and cheaper than filtration, producing in every respect results equally satisfactory, and applicable to the water supply of many cities, both great and small.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1068.

REINFORCED CONCRETE FOUNDATIONS OVER EXCAVATIONS ON PAVED STREETS.

By John McNeal, M. Am. Soc. C. E.

Constant excavations on the paved streets of Easton, Pa., and numerous depressions formed over previously excavated trenches, have called the writer's attention to the necessity of providing some means for protecting the surface of the street from gradual settlement below the general level of the adjoining paving.

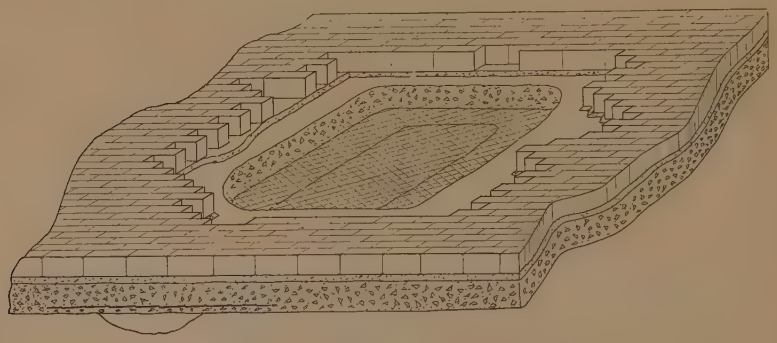
No matter how carefully the back-filling of a trench is done, there is bound to be a settlement of the earth replaced in the trench, and in some cases this settlement has not begun to show for a year or more after the paving has been replaced.

In order to avoid settlement of the surface, the foundation for repaving over all trenches has been reinforced with expanded metal. For a trench not more than 20 in. in width, the writer uses expanded metal, of 3-in. mesh and No. 10 gauge steel, having a cross-sectional area of not less than 0.185 sq. in. per ft. in width.

After the trench is properly back-filled, the old concrete is removed for a distance of at least 1 ft. beyond each side of the trench, and expanded metal is placed over the entire opening. A thickness of 6 in. of concrete, mixed in the proportion of 1 part Portland cement, 3 parts sand, and 5 parts broken stone, is placed over and around the expanded metal in such a manner that the metal is embedded in it, $1\frac{1}{2}$ in. above the base and $4\frac{1}{2}$ in. below the surface.

After the concrete is thoroughly rammed to an even surface, the paving material is replaced on this surface to conform with the adjoining paving.

These requirements met with some opposition at first from corporations and local contractors who are constantly digging up the paved streets for new mains and repairs, but now all excavated trenches are repaved with the expanded metal reinforcement.



REINFORCED CONCRETE FOUNDATIONS
OVER
EXCAVATIONS ON PAVED STREETS.

FIG. 1.

The cost of this foundation per linear foot of trench is about 37½ cents more than that of plain concrete.

Assuming the working strength of the steel to be 20 000 lb. per sq. in., and allowing for the shape of the metal, which distributes the load over 20 in. of trench, a wheel load of from 650 to 700 lb. would be supported safely without the aid of the back-filling below the foundation.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1069.

INVAR (NICKEL-STEEL) TAPES ON THE MEASUREMENT OF SIX PRIMARY BASE LINES.*

By Owen B. French, M. Am. Soc. C. E.

With Discussion by Messrs. J. A. Ockerson, Horace Andrews, Noah Cummings, and Owen B. French.

Until near the end of the last century, geodesists considered some form of bar apparatus necessary for the accurate measurement of primary base lines. The use of any bar apparatus is very expensive, hence, when it became apparent that frequent bases were required in a triangulation scheme, a more economical form of measuring apparatus was necessary. As the question of economy would undoubtedly be solved by the use of long tapes or wires, many attempts were made to devise some means of securing a satisfactory degree of accuracy with apparatus of this type. Long steel tapes or wires had been in use for many years for measuring purposes, but, until about 1890, they were not considered accurate enough for primary measurement. Although many had investigated the use of tapes for accurate measurement, to a greater or less extent, one of the first to make a successful investigation was Professor R. S. Woodward, while he was an Assistant on the United States Coast and Geodetic Survey.

The results of his work and their discussion, together with the mathematical formula required for the general use of tapes, may be

*Presented at the meeting of December 4th, 1907.

found in Appendix 8 of the Coast and Geodetic Survey Report for 1892.*

Previous Work with Steel Tapes.—The long steel tapes used by Professor Woodward in his investigations have been used since in the measurement of a number of base lines by the Coast and Geodetic Survey. The most important work of this character was the measurement of nine base lines along the 98th meridian, by Mr. A. L. Baldwin, in 1900.† A direct comparison between the measurements with bars and steel tapes was made on each of the nine bases. The duplex bar apparatus—one of the best forms of bar apparatus known—was used, and also both 50 and 100-m. steel tapes. The results of this work demonstrated that steel tapes, when used at night, and standardized under the same conditions that prevail during the base measures, gave practically the same degree of accuracy as the duplex apparatus, for about one-third of the cost. This work also demonstrated that the gain, either in accuracy or economy, by the use of a tape more than 50 m. long, was so small as to fail to offset the gain in convenience of manipulation in the field possessed by the shorter tape. Consequently, the Coast and Geodetic Survey, until recently, has considered the 50-m. steel tape the best form of base-measuring apparatus, if used at night, when the temperature of the tape can be obtained very accurately.

Invar, or Nickel-Steel Alloy.—When, a few years ago, Dr. C. E. Guillaume, Assistant Director of the International Bureau of Weights and Measures, near Paris, discovered an alloy of nickel and steel which possessed a small coefficient of expansion, its use for base measurement was immediately suggested, as the effect of temperature errors would undoubtedly be greatly reduced by using such a metal. This alloy is made near Paris by a secret process, and contains about 36% of nickel. It has been given the name of "invar," a word derived from the same root as the word invariable, and having a similar meaning.

Guillaume's investigations of the properties of this alloy have been quite extensive. The results of his work have been published from time to time in such scientific publications as *Bulletins des Séances de la Société Française de Physique, Archives des Sciences, Comptes Rendus, Journal de Physique, Metallographist,* etc., in addition to the publications of the International Bureau of Weights and Measures.

* "On the Measurement of the Holton Base, Holton, Ripley County, Indiana, and the St. Albans Base, Kanawha County, West Virginia."

† "On the Measurement of Nine Bases Along the Ninety-eighth Meridian." Apx. 3, U. S. Coast and Geodetic Survey Report, 1901, p. 229.

Guillaume's investigations extend over a period of several years, and, in so far as they apply to base measurement, have been made with bars, or wires usually 24 m. long.

U. S. Coast and Geodetic Survey Invar Tapes.—The use of wires instead of tapes has not been considered advisable on the Coast and Geodetic Survey, the objections to wires being considered of more consequence than the objections to tapes.

As soon as Guillaume's investigations with the invar wires had advanced far enough to prove that the metal is comparatively stable, the Coast and Geodetic Survey decided to test some tapes of this alloy. Although invar wires could be obtained with little difficulty, no invar tapes could be secured until December, 1905, when they were purchased in London, England.

These invar tapes are about 52 m. (171 ft.) long, 6.3 mm. (0.25 in.) wide, and 0.5 mm. (0.02 in.) thick, with a mass of about 25 g. per m. of length (0.027 oz. per ft. of length). They are slightly longer and heavier than the steel tapes used by the U. S. Coast and Geodetic Survey, the mass of the latter being about 20 g. per m. of length.

The invar tapes are considerably brighter than the steel tapes, being more nearly like nickel in appearance. Although they are less easily oxidized than steel tapes, they require almost as much care in order to keep them free from rust. The metal is softer, more easily bent, and less elastic than steel.

When these tapes were laid, without tension, upon a flat surface, they appeared crooked, being full of small bends in all directions. These bends were not large enough, however, to cause any part of the tape to be more than about $\frac{1}{2}$ in. from a straight line. The large bends were removed from all tapes used for base measurement, before beginning the standardization.

When the tapes were stretched under a tension of 15 kg. (the tension used on all the measures), the remaining small bends were practically eliminated, although in certain lights numerous minute irregularities were distinctly visible. These small irregularities were so nearly the same every time the tape was stretched under a tension of 15 kg., that the length of the tape was practically constant. This was proven in the standardization of these tapes.

The measurement of several primary base lines had been postponed a year or more by the Coast and Geodetic Survey, in hope of

securing some of these invar tapes; hence, when they were received. preparations for the measurement of these bases were immediately begun.

Several of the invar tapes were prepared for measuring purposes, in the same manner as the steel tapes, by the Instrument Division of the Coast and Geodetic Survey. Small silver sleeves were riveted rigidly to each tape near either end, to carry the graduation marks, the distance between the two marks on a tape being very nearly 50 m.

An aluminum reel, 16 in. in diameter, was made for each tape. The size of these reels was not considered of sufficient consequence to warrant special investigation, hence it was fixed in accordance with a statement made by Mr. Baugh when transmitting the tapes.

The fact that these reels are not small enough to affect the length of a tape was proved during the standardization by reeling and unreeling one tape one hundred times on several different days, no change in its length being noticeable.

Investigations of the Properties of Invar Tapes.—Notwithstanding the fact that Guillaume's investigations with invar wires were quite extensive and satisfactory, it was considered best to make experiments enough with these tapes to be certain that they did not possess properties which would affect materially the accuracy of any measures that might be made with them. Consequently, several short pieces of the tapes were tested, at the National Bureau of Standards, in order to find:

First.—The tensile strength;

Second.—The yield point; and

Third.—Whether the continued application and removal of small loads affected the length of the tape.

The tensile strength was found to be about 100 000 lb. per sq. in. (practically one-half that of ordinary tapes), a load of more than 450 lb. being required to break the specimens. The yield point, determined with a Henning mirror extensometer, was about 70% of the tensile strength.

One specimen had a load of 40 lb. suspended from it for 60 hours, without showing any change in length. Loads up to 60 lb. were applied and removed on one of the specimens many times, the lengths corresponding to the same loads being practically identical.

The results of these experiments were very satisfactory; and no

properties derogatory to the use of these tapes for measuring purposes were discovered.

Disadvantages of Steel Tapes.—Steel tapes must be standardized in the field and used at night in order to obtain the accuracy required for the measurement of primary base lines. The cost of the measurement is increased very considerably on this account. As these conditions are necessary, owing to the uncertainty in the determination of the temperature of the tape, it was hoped that this expense could be eliminated by using a metal which possessed a small coefficient of expansion. Consequently, the invar tapes were standardized at the Bureau of Standards both before and after the field measures.

Night measures are objectionable for many reasons: The necessity for illumination causes a loss in accuracy and also a loss in speed, thus increasing the cost of the work. Therefore, the invar tapes were used during daylight hours only.

Owing to the fact that the invar tapes were unknown, and that there was a possibility of their proving too unstable for accurate work, it was decided to make a complete measurement of all the bases with the steel tapes as well as with the invar tapes, thus insuring a satisfactory determination of the lengths of the bases and affording a means of studying the relative action of the two kinds of tapes.

Standardization.—The steel tapes were used entirely at night and standardized in the field just before and just after the measurement of the six bases. The invar tapes were standardized at the Bureau of Standards.

All the tapes were standardized in practically the same manner, by comparing them many times with a 50-m. comparator, the length of which was obtained with the iced-bar apparatus.

The 50-m. comparator used at the Bureau of Standards is located in a tunnel about 2½ m. high and wide, and 52 m. long. Along one wall of this tunnel are a number of pipes through which brine, at a temperature of —10°, may be pumped to get low temperatures. The iced bar used for this work is the one designed by Professor Woodward, and described in the Coast and Geodetic Survey Report for 1892. It is a steel bar, a little more than 5 m. long, the measure being the distance between two fine lines cut into platinum iridium plugs, one near each end of the bar. The bar is supported by adjustable bolts in a V-trough, the trough resting upon trucks also adjustable. A

track was built in the floor of the tunnel for these trucks to run upon so that the bar could be moved along rapidly. Microscopes for holding the measure while the bar was being run ahead were mounted upon heavy iron arms, one end of each being attached, in an adjustable manner, to the top of a stone post. These posts were 5 m. apart, the length of the bar. A hole was cut through the top of each post to permit the light from an electric bulb, suspended back of the post, to supply the illumination for the microscope. Each stone post was mounted upon a concrete pier.

The microscopes were carefully aligned by a fine wire stretched between the end microscopes and near the foci of all. The heights of the microscopes were such that their foci were nearly in a straight line which was practically horizontal.

The inclination of the bar in each position (the end marks of the bar being in the foci of the microscopes) was determined with a sector attached to the side of the trough, thus giving data for the elimination of the effect of the small differences in height between the foci of the microscopes.

The ends of the comparator were marked by spherical-headed bolts set in the concrete piers which support the end microscope posts. These marks were directly under the end microscopes, which were referred to the end marks frequently during the standardization by the cut-off cylinders used heretofore.

The bar was aligned and leveled in the usual manner, the trough being kept as full of crushed ice as possible during each operation. The bar was usually surrounded with ice at least a half hour before beginning the measurement.

A determination of the length of the comparator consisted of at least two measures with the iced bar in opposite directions.

When making the tape comparisons at the Bureau of Standards, a determination of the length of the comparator was made at the beginning and also at the end of each day's work. When at work in the field such a determination was made before beginning the tape comparisons, after their completion, and at intermediate times, if necessary to make the interval between the tape comparisons and comparator determination less than 24 hours.

The lengths of the tapes were obtained by stretching them under the end microscopes of the comparator, always supporting and hand-

ling them in practically the same manner as they were to be used in the field. The invar and steel tapes were handled in exactly the same manner. The tape was supported at the graduation marks and also at one point in the middle in line with the end supports. A tension of 15 kg. was used, indicated by a spring balance having a dial with a hand working over its face. A complete revolution corresponded to 5 kg. The tension could be read to about 10 g. without difficulty.

A thermometer with a metal back was fastened 1 m. from the graduation mark (and toward the middle), at each end of the tape. These were always read immediately after making pointings upon the graduation marks of a tape.

During standardization the rear end of the tape was held rigidly by an adjustable clamp. The forward or balance end was held by an adjustable ratchet device, at the Bureau of Standards, and by the tape stretcher used on the base measurement, for the field standardization.

A determination of the length of the tape consisted of two simultaneous pointings by the two observers on the graduation marks of the tape, followed immediately by two more after the observers had exchanged places.

An ordinary day's work was found to be a double comparison of from six to eight tapes, with an iced-bar determination of the length of the comparator, both before and after. During February, 1906, before beginning the base measures, twelve or thirteen comparisons were obtained for each of the six invar tapes, six comparisons at a temperature of about + 4° cent., and the remainder at about + 25° cent. After the base measures, in October, 1906, five of these tapes were again standardized, four comparisons being obtained for each.

The results of the standardization for the five invar tapes are as follows, each temperature being the mean of all the observed temperatures for that tape, as the length is the mean of all the observed lengths:

	Temperature, in degrees, cent.	Length, in millimeters, over 50 m.	
Tape No. 437.			
February, 1906.....	14.24	+ 7.816	
October, 1906.....	25.62	+ 8.079	
Mean.....	19.93	7.947	± 0.007 mm.

	Temperature, in degrees, cent.	Length, in millimeters, over 50 m.	
Tape No. 438.			
February, 1906.....	14.26	+ 8.057	
October, 1906.....	25.60	+ 8.380	
Mean.....	19.93	+ 8.218	± 0.009 mm.
Tape No. 439.			
February, 1906.....	14.24	+ 6.578	
October, 1906.....	25.60	+ 6.864	
Mean.....	19.92	+ 6.721	± 0.007 mm.
Tape No. 440.			
February, 1906.....	14.38	+ 9.418	
October, 1906.....	25.70	+ 9.702	
Mean.....	20.04	+ 9.560	± 0.008 mm.
Tape No. 442.			
February, 1906.....	14.16	+ 9.218	
October, 1906.....	25.45	+ 9.548	
Mean.....	19.80	+ 9.383	± 0.010 mm.

Table 1 shows the individual values for Tape No. 440, and is a fair sample of the manner in which the results agree:

TABLE 1.—INDIVIDUAL VALUES OF INVAR TAPE NO. 440.

Date.	Temperature in degrees, cent.	Observed length over 50 m.+, in millimeters.	Computed length over 50 m.+, in millimeters.	Computed minus observed length, in millimeters.
1906.				
Feb. 10.....	27.3	+ 9.666	+ 9.696	+ 0.030
" 12.....	4.4	9.208	9.268	+ 0.060
" 12.....	4.3	9.199	9.266	+ 0.067
" 13.....	3.9	9.195	9.258	+ 0.063
" 13.....	4.1	9.186	9.262	+ 0.076
" 15.....	3.3	9.225	9.247	+ 0.022
" 15.....	2.2	9.287	9.227	— 0.060
" 17.....	24.5	9.587	9.643	+ 0.056
" 17.....	24.6	9.615	9.645	+ 0.030
" 19.....	25.7	9.650	9.666	+ 0.016
" 19.....	24.6	9.616	9.645	+ 0.029
Mar. 1.....	24.1	9.600	9.636	+ 0.036
" 1.....	24.6	9.596	9.645	+ 0.049
Mean.....	14.38	+ 9.418	.....	.....
Oct. 4.....	25.7	+ 9.716	9.666	— 0.050
" 4.....	25.8	9.686	9.668	— 0.018
" 5.....	25.4	9.685	9.660	— 0.025
" 5.....	25.9	+ 9.721	9.670	— 0.051
Mean.....	25.70	+ 9.702		
Final Mean.....	20.04	+ 9.560 ± 0.008		

The quantities in the column headed "Computed" were obtained from the final mean by reducing it to the temperature of the individual values. The probable error was computed from the residuals given in the last column.

By reducing all the values obtained in the February standardization to the temperature of the October standardization, it was found that there was a slight lengthening of all the tapes during this period. However, as there are residuals of both signs in the February work, at least a part of this difference may be apparent rather than actual. It averages about 1 part in 800 000 for the five tapes, the greatest being 1 part in 617 000. If this change took place uniformly, the mean of the two standardizations gives values from which the lengths of the bases are obtained very accurately. Even if it were a sudden change, the error on any base from this source could not be more than 1 part in 1 600 000, and might be much less. It was probably a more or less gradual change, as there was nothing in the measures to indicate any sudden change. It may have been due to a change in the molecules of the tape itself, but is more likely to have been a straightening out of the many small bends in the tapes. If the latter, it may have been either a gradual change or a sudden one, although the former is the more probable.

Tapes Nos. 437 and 442 were never used, except for the comparisons. Tape No. 437 was taken into the field, but was not used on the base measures. Tape No. 442 was not even taken into the field. The large bends were not removed from Tape No. 442 before standardizing it, and the graduation marks were very irregular, hence its change is probably due partly to these two conditions. As the largest change was only 0.08 mm. per tape length, a quantity barely perceptible to the naked eye, it is certainly not of serious consequence in any case. Steel tapes have been found to change length more than this.

Coefficients of Expansion.—The coefficients of expansion for these tapes were computed from the February observations, the large range in temperatures having been secured for this purpose. During this work at the Bureau of Standards, four steel tapes were also compared for the purpose of getting their coefficients of expansion. Table 2 shows the results obtained for all the tapes:

These coefficients for the invar tapes are very small, being practically one-twenty-eighth of those of steel tapes. They are about one-

half of those of the wires used in the base measurement through the Simplon Tunnel, by Guillaume, thus showing that they are probably smaller than those usually obtained for this alloy.

TABLE 2.

Tape Number.	Expansion per degree centigrade per 50 m.	Probable Error.	Coefficient per unit length per degree centigrade.
Invar No. 437	0.0207	±0.0009	0.000 000 41
" " 438	0.0213	0.0008	0.000 000 43
" " 439	0.0203	0.0006	0.000 000 41
" " 440	0.0187	0.0006	0.000 000 37
" " 442	0.0220	0.0009	0.000 000 44
Steel No. 403	0.568	±0.003	0.000 011 4
" " 404	0.568	0.003	0.000 011 4
" " 405	0.569	0.004	0.000 011 4
" " 406	0.565	0.003	0.000 011 3

Bases Measured.—Early in March, 1906, the party left Washington, D. C., to measure six base lines. The first measured was Point Isabel Base, in Southern Texas, near the mouth of the Rio Grande River; second, Willamette Base, in the Willamette Valley, Oregon; third, Tacoma Base, near Tacoma, Wash.; fourth, Stephen Base, near Stephen, Minn.; fifth, Brown Valley Base, South Dakota, near Brown Valley, Minn.; and, last, Royalton Base, near Royalton, Minn. The first, fourth and fifth bases control part of the triangulation along the 98th meridian; the two on the Pacific Coast control the main scheme along that coast between the 39th-parallel triangulation and the Puget Sound work. The last base controls, in part, the triangulation which connects the 98th-meridian scheme with the triangulation of the Great Lakes near Duluth.

All six bases were measured with three steel tapes and also with three invar tapes, each tape being used on about two-thirds of each base. Two complete measures of each base were made with the steel tapes and two with the invar, the steel tapes being used at night only and the invar in the daylight. The measures were arranged so that a complete inter-comparison of all the tapes was obtained on each base.

Field Standardization.—The steel tapes were standardized at the first and also at the last base measured. A 50-m. comparator was built at each of these places, and its length was determined with the iced bar in the same manner as at the Bureau of Standards. The method

of making the tape comparisons was the same as that used at the Bureau of Standards, except that the regular tape stretcher used in the base measures was used at the forward end on the field standardization. Two comparisons were obtained for each of the steel tapes, on each of two nights, at both standardizations, or eight comparisons in all for each tape.

Adopted Lengths of All Tapes.—The adopted lengths and other data for the steel tapes, as derived from the field comparisons, are given in Table 3. The coefficient of expansion was obtained from the work at the Bureau of Standards. The table also gives the adopted lengths and other data for the invar tapes as obtained from the observations at the Bureau of Standards, which have been previously described.

TABLE 3.

STEEL TAPES.

	mm.	mm.	Degrees.
T_{248} = 50 m.	+ 3.51	+ 0.527	$(t - 20.62)$
	± 0.010	± 0.003	
T_{403} = 50 m.	+ 12.12	+ 0.568	$(t - 20.59)$
	± 0.024	± 0.003	
T_{405} = 50 m.	+ 12.56	+ 0.569	$(t - 20.74)$
	± 0.017	± 0.004	
T_{406} = 50 m.	+ 12.34	+ 0.565	$(t - 20.88)$
	± 0.021	± 0.003	

INVAR TAPES.

	mm.	mm.	Degrees.
T_{437} = 50 m.	+ 7.947	+ 0.0207	$(t - 19.93)$
	± 0.007	± 0.0009	
T_{438} = 50 m.	+ 8.218	+ 0.0213	$(t - 19.93)$
	± 0.009	± 0.0008	
T_{439} = 50 m.	+ 6.721	+ 0.0203	$(t - 19.92)$
	± 0.007	± 0.0006	
T_{440} = 50 m.	+ 9.560	+ 0.0187	$(t - 20.04)$
	± 0.008	± 0.0006	
T_{442} = 50 m.	+ 9.383	+ 0.0220	$(t - 19.80)$
	± 0.010	± 0.0009	

Field Procedure.—The preparation of the base lines for measurement was practically the same as for previous base measurements with steel tapes. Posts, 4 by 4 in., were driven every 50 m. along the line, with posts 2 by 4 in. midway between them. A nail, for the middle support of the tape, was driven into each of the 2 by 4-in. posts, in line

with the tops of the adjoining 4 by 4-in. posts, or raised enough so that the tape would clear the ground or other objects. In this case its height was determined while running the levels over the base. All these posts were carefully aligned with a theodolite. Forward sights only were used, and posts were rarely set on more than 300 m. from one position of the instrument. There was probably no tape length that was out of parallelism to the base by as much as 1 cm., thus eliminating errors from this source.

A forward and backward line of levels was run over each base to determine the height of each 4 by 4-in. post and all 2 by 4-in. posts which were not on grade with the tops of adjoining 4 by 4-in. posts. The difference in height of any two adjoining posts was probably obtained within 1 cm., hence errors from this source are also negligible. The corrections for grade were obtained from a table prepared for the argument, difference in height, h, using the formula:

$$c = -\frac{h^2}{2s} - \frac{h^4}{8s^3} - \frac{h^6}{16s^5} -, \text{ etc.},$$

s in this case being 50 m.

Copper strips, of the same thickness as the graduation sleeves on the tapes, were nailed to the tops of the 4 by 4-in. posts to hold the marks for the measures. Each base was divided into sections about 1 km. in length. The various measures of a section were all referred to the same lines on the copper strips of the terminal posts, hence are directly comparable.

Heavier posts were set very solidly at the ends of the sections to hold the measure for several days, if necessary. Only complete sections were measured at one time, thus the 4 by 4-in. posts were not depended upon to hold the measure for more than a few minutes.

The tapes were stretched over the 4 by 4-in. posts, with a thermometer attached to the top, in the same manner as in the standardization. The rear end was held by a small steel bar. The forward end was attached to the hook of the balance, which was fastened to the stretcher by a gimbal attachment with counterpoise, so that the balance could always be kept horizontal. A steel point was fastened at the lower end of the stretcher bar, and a quick and accurate adjustment for the height of the tape was made without using the screw adjustment, by merely pressing the stretcher into the ground more or less as might be required.

All 4 by 4-in. posts were set at a uniform height above the ground (about ½ m.), so that no time would be lost during the measure in adjusting the tape stretcher.

Party Organization.—The measuring party was composed of two observers, two tape stretchers, one recorder, and two men to support the middle of the tape while it was being carried forward, seven in all. The first three bases were prepared by the measuring party, but the last three were prepared by a separate party, consisting of five men. This party prepared three bases, aggregating a length of 27 km., in 17 working days. This time includes delays on account of weather and travel between bases. The latter required four days. On about a kilometer of one base, the posts were set in a pond where the water was about 2 ft. deep, and each post in the water was double braced in order to insure its stability, thus delaying the preparations very materially. This party had no difficulty in preparing between 3 and 4 km. per day, under ordinary conditions. When the preparation party finished with a base, everything was ready for beginning the measures. The copper marking strips were usually nailed to the tops of the posts during the first measure.

Speed of Measurement.—A single measurement of 12 km. was found to be an ordinary day's work, when half of it was made at night. Occasionally, 14 km., or more, were measured in one day. This, however, occurred only when the work had been delayed from some cause, and was too much to be maintained for any length of time. The speed with which the measurement progressed was a trifle more rapid with the invar than with the steel tapes. After the party acquired some experience, a speed of more than 2 km. per hour was ordinarily maintained. The average speed for the invar measures on the last base was 2.6 km. per hour, the whole party being well trained by this time. No attempt was made merely to acquire speed, a tape length never being marked until everything was apparently steady and correct.

Record and Computation.—The record kept on this work was very simple. It consisted principally of two thermometer readings for each tape length, with an occasional set-up or set-back, made in order to keep the marks on the copper marking strips. Notes as to the condition of the weather, or the occurrence of anything that might affect the measures, were inserted whenever needed. In making the computations the average temperature for each section was used to deter-

mine the average length of the tape during the measurement of that section.

Table 4 gives the results of the measures for each section of the Brown Valley Base, and is a fair sample of the form of computation and differences between the results for the individual sections, as obtained on all the bases.

In Table 4 the time is given in hours and tenths. All measures with steel tapes were made at night, and all measures with invar tapes in the daytime, hence it is unnecessary to insert A. M. or P. M. The columns headed "Weather," "Wind," "Temperature," "Temperature, rising or falling," and "Temperature range" are inserted to permit the study of possible sources of errors. The abbreviations used in these columns are: R = rising; F = falling, for temperature; F = fair; C = cloudy; P = Partly cloudy; R = rain; M = mist; and D = dew, for weather conditions; L = light; M = moderate; and H = fresh, for wind, with the usual abbreviations for its direction. The temperature range is given to tenths of degrees for the steel tapes, but only to degrees for the invar tapes, owing to their smaller coefficients of expansion.

Column 6 shows the average temperatures for each measurement of a section, corrected for thermometer errors. Column 11 gives the corrections to the tapes due to differences between this temperature of observation and that of standardization for the particular tape. Column 12 gives the grade corrections as obtained from the difference in heights between adjoining posts, as previously mentioned in this paper. Column 13 gives the reduction to sea level obtained from the average height of the section, h, using the formula

$$c = -s\frac{h}{\rho} + s\frac{h^2}{\rho^2} - s\frac{h^3}{\rho^3} +, \text{ etc.},$$

s being the length of the section, and ρ the radius of curvature of the earth for this base line. Column 14 gives the algebraic sums of the set-ups and set-backs for each measure of the sections. Column 15 gives the tape corrections for the sections, obtained by multiplying the excess of the length of the tape over 50 m. by the number of tape lengths in the section. Column 16 shows the algebraic sum of Columns 11 to 15. These quantities are the results of the individual measures for the various sections. Column 17 shows the lengths of the sections as determined with the steel tapes, and Column 18, the lengths as

determined with the invar tapes. Column 19 gives the residuals for the various measures of the sections, not combining the invar and steel results. Column 20 shows the differences between the invar and steel measures of the various sections.

Computation of Probable Errors.—The probable error for the base was determined as follows: The probable error of each section was determined from the residuals of Column 19, keeping the measures with the steel and invar tapes separate. These values were combined with the probable errors of the tapes, the latter being obtained by multiplying the probable error of one length of the tape, given with the adopted lengths of tapes, by the number of tape lengths in the section.

The probable error of the length of the base, given by the steel tapes, was obtained by taking the square root of the sum of the squares of the probable errors of all the sections obtained from the measures with steel tapes, and the probable error of the length from the measures with the invar tapes, by a similar combination of the probable errors of the sections obtained from measures with invar tapes.

Probable error of measures with steel tapes, Brown Valley Base ..... ± 4.98 mm.

Probable error of measures with invar tapes, Brown Valley Base ..... ± 2.05 mm.

These values were then combined with the probable errors due to the probable errors of the coefficients of expansion. The latter were obtained by taking the sum of all the temperature differences between the observed and standard temperatures of the tapes on the base and multiplying this by the average probable error of the coefficients of expansion of the tapes, these probable errors being nearly equal.

The probable error due to probable errors of coefficients of expansion for steel tapes was..... ± 2.46 mm.

The probable error due to probable errors of coefficients of expansion for invar tapes was..... ± 0.22 mm.

Combining these with the values already given we have:

Probable error due to measurements and coefficients:

Steel tapes ..... ± 5.55 mm.
Invar tapes ..... ± 2.06 mm.
Weighted mean, steel and invar tapes..... ± 2.09 mm.

TABLE 4.—RESULTS OF MEASURES

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Number of section and posts.	Date.	Time of day.	Direction of measure.	Number of tape.	Mean Temperature.	Temperature, Range.	Temperature, Rising or Falling.	Weather.	Wind.	Temperature.
	1906.	Hour.			Degrees.	Degrees.				m.
1	June 27	9.7	SE.	440	24.33	6	RF	P	LS	+0.0016
S. E. B. to 20...	June 27	10.2	NW.	438	25.28	5	RF	P	LS	+0.0023
	June 27	9.0	SE.	406	21.05	0.6	RF	PD	.....	+0.0019
	June 27	9.5	NW.	403	21.20	0.9	F	PD	.....	+0.0069
2	June 27	9.1	SE.	440	22.77	6	R	P	LS	+0.0010
20 to 40.....	June 27	10.7	NW.	438	26.40	6	RF	P	LS	+0.0028
	June 27	8.5	SE.	406	20.39	1.4	FR	PD	.....	−0.0055
	June 27	10.0	NW.	403	20.10	1.0	F	PD	.....	−0.0056
3	June 26, 27	.....	SE.	440	21.15	3	RF	P	LN	+0.0004
40 to 60.....	June 27	11.0	NW.	438	24.60	3	F	P	LN	+0.0013
	June 27	3.2	NW.	439	24.46	3	RF	P	LN	+0.0006
	June 29, 27	.....	SE.	406	19.68	2.0	F	PD	.....	−0.0136
	June 29	10.5	NW.	403	19.57	0.6	FR	PD	.....	−0.0076
	June 29	9.0	NW.	405	15.98	0.6	RF	PD	.....	−0.0190
4	June 26	4.0	SE.	440	22.90	5	FR	P	LN	+0.0011
60 to 80.....	June 26	3.6	NW.	439	25.02	5	RF	P	LN	+0.0021
	June 29	9.9	SE.	406	14.72	3.0	RF	DF	LE	−0.0696
	June 29	9.5	NW.	405	13.69	4.1	F	FD	LE	−0.0802
5	June 26	4.4	NW.	439	25.50	5	RF	P	LN	+0.0023
80 to 100.....	June 26	5.2	SE.	440	24.82	3	RFR	P	LN	+0.0018
	June 29	10.0	NW.	405	12.53	1.7	FR	FD	LE	−0.0934
	June 29	11.0	SE.	406	13.16	1.8	FR	FD	LE	−0.0872
6	June 26	4.6	NW.	439	24.45	3	R	P	LN	+0.0005
100 to 105.....	June 26	5.0	SE.	440	25.46	2	F	P	LN	+0.0005
	June 29	10.3	NW.	405	12.02	0.4	R	FD	LE	−0.0222
	June 29	10.5	SE.	406	12.76	0.5	R	FD	LE	−0.0207
7	June 26	9.0	NW.	438	18.28	3	FR	P	L	−0.0007
105 to 125.....	June 26	11.4	SE.	439	20.71	6	RF	P	L	+0.0003
	June 30	8 5	NW.	405	11.20	1.4	FR	FD	LNE	−0.1086
	June 30	10.8	SE.	403	8.83	1.5	FR	FD	LNE	−0.1336
8	June 26	9.5	NW.	438	19.66	6	RF	P	L	−0.0001
25 to 145.....	June 26	10.9	SE.	439	22.32	7	F	P	L	+0.0010
	June 30	8.9	NW.	405	10.47	1.3	FR	FD	LNE	−0.1169
	June 30	10.4	SE.	403	9.52	1.3	FR	FD	LNE	−0.1258
9	June 26	9.9	NW.	438	20.96	5	RF	P	L	+0.0004
145 to N. W. B...	June 26	10.5	SE.	439	22.34	6	FR	P	L	+0.0010
	June 30	9.4	NW.	405	9.56	0.9	FR	FD	LNE	−0.1272
	June 30	9.9	SE.	403	9.64	1.0	FR	FD	LNE	−0.1244

* This section was measured in part with each of the tapes given.
† Full or half-tape lengths did not fit between the ends of this section, hence, the fractional

OF BROWN VALLEY BASE.

Corrections.								
(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
Grade.	Sea Level.	Set-up or Set-back.	Tape Length.	Corrected length of section.	Mean Length, Steel tapes.	Mean Length, Invar tapes.	Residual, Mean minus Observed.	Difference, Invar minus Steel.
m.	m.	m.	m.	m.	m.	m.	mm.	mm.
−0.0805	−0.0542	+0.0389	+0.1912	1 000.0970	.....	.....	−0.3	.....
.....	.....	+0.0644	+0.1644	.0964	.....	1 000.0967	+0.3	.....
.....	.....	−0.0113	+0.2468	.1027	.....	.....	−1.5	.....
.....	.....	−0.0148	+0.2424	.0998	1 000.1012	.....	+1.4	−4.5
−0.0143	−0.0538	−0.0843	+0.1912	1 000.0398	.....	1 000.0397	−0.1	.....
.....	.....	−0.0595	+0.1644	.0396	.....	.....	+0.1	.....
.....	.....	−0.1335	+0.2468	.0397	.....	.....	+0.3	.....
.....	.....	−0.1285	+0.2424	.0402	1 000.0400	.....	−0.2	−0.2
−0.0130	−0.0541	−0.1290	+0.1912	999.9955	.....	.....	0.0	.....
.....	.....	−0.0932 *	+0.1069 *	.9955	.....	999.9955	0.0	.....
.....	.....	.....	+0.0470 *	.....	.....	.....	.....	.....
.....	.....	−0.1679	+0.2468	.9982	.....	.....	−1.1	.....
.....	.....	−0.1559 *	+0.1576 *	.9959	.....	.....	+1.2	.....
.....	.....	.....	+0.0879 *	.....	999.9971	.....	.....	−1.6
−0.0126	−0.0541	−0.0292	+0.1912	1 000.0964	.....	.....	+0.9	.....
.....	.....	+0.0284	+0.1344	.0982	.....	1 000.0973	−0.9	.....
.....	.....	−0.0166	+0.2468	.0939	.....	.....	−0.3	.....
.....	.....	−0.0110	+0.2512	.0943	1 000.0936	.....	+0.3	+3.7
−0.0103	−0.0543	+0.0215	+0.1344	1 000.0931	.....	.....	−1.0	.....
.....	.....	−0.0369	+0.1912	.0910	.....	1 000.0921	+1.1	.....
.....	.....	−0.0026	+0.2512	.0901	.....	.....	+0.5	.....
.....	.....	−0.0034	+0.2468	.0911	1 000.0906	.....	−0.5	+1.5
−0.0061	−0.0122	+0.0066	+0.0303	222.9643†	.....	.....	+0.2	.....
.....	.....	−0.0057	+0.0430	.9647	.....	222.9645†	−0.2	.....
.....	.....	+0.0024	+0.0565	.9636†	.....	.....	0.0	.....
.....	.....	+0.0018	+0.0556	.9636	222.9635†	.....	0.0	+0.9
−0.0298	−0.0544	+0.0297	+0.1644	1 000.1092	.....	.....	+1.8	.....
.....	.....	+0.0623	+0.1344	.1128	.....	1 000.1110	−1.8	.....
.....	.....	+0.0464	+0.2512	.1048	.....	.....	+2.9	.....
.....	.....	+0.0860	+0.2424	.1106	1 000.1077	.....	−2.9	+3.3
−0.0419	−0.0547	+0.0359	+0.1644	1 000.1036	.....	.....	+1.9	.....
.....	.....	+0.0686	+0.1344	.1074	.....	1 000.1055	−1.9	.....
.....	.....	+0.0633	+0.2512	.1010	.....	.....	+3.3	.....
.....	.....	+0.0876	+0.2424	.1076	1 000.1043	.....	−3.3	+1.2
−0.0642	−0.0553	+0.0222	+0.1644	1 000.0675	.....	.....	0.2	.....
.....	.....	+0.0512	+0.1344	.0671	.....	1 000.0673	+0.2	.....
.....	.....	+0.0614	+0.2512	.0659	.....	.....	+5.4	.....
.....	.....	+0.0783	+0.2424	.0768	1 000.0713	.....	−5.5	−4.0

tape length (−2.0548), measured with a 3-m. bar, has been included in these lengths.

The probable errors given so far do not include the probable error in the determination of the length of the comparator. This was practically ± 0.014 mm. As the values for the tapes depend upon two standardizations, we can take $\frac{1}{2}\sqrt{2\,(0.014)^2} = \pm\ 0.010$ mm. as the probable error per tape length, resulting from the uncertainty in the length of the comparator.

As there were 164 tape lengths in the Brown Valley Base, the effect of the probable error of the comparator is $164 \times \pm\ 0.010 = \pm\ 1.64$ mm. This, combined with the last value obtained, gives the final probable error of the Brown Valley Base, ± 2.83 mm. or 1 part in 2 910 000.

This includes all known sources of error throughout all the operations necessary to obtain the length of the base in terms of the international meter.

Length of Bases.—The final lengths of the six base lines, together with their logarithms and probable errors, all computed in the same manner as described above, are given in Table 5.

TABLE 5.

Base.	Length.		Probable error.
	Meters.	Logarithm.	
Point Isabel.....	7 384.9220	3.8683459 ± 2	±3.02 mm., or 1 part in 2 450 000
Willamette.....	14 019.3781	4.1467287 ± 1	4.09 " " 1 " " 3 430 000
Tacoma.....	12 055.5701	4.0811877 ± 2	3.99 " " 1 " " 3 020 000
Stephen.....	9 221.8333	3.9648173 ± 2	4.25 " " 1 " " 2 170 000
Brown Valley.....	8 223.5695	3.9150604 ± 1	2.83 " " 1 " " 2 910 000
Royalton.....	9 637.5508	3.9839667 ±	3.24 " " 1 " " 2 980 000
Average.....	10 090		1 part in 2 760 000
Total.....	60 543		

Comparison Between Results for Steel and Invar Tapes.—In order to afford a ready means for comparing the results obtained with the invar and steel tapes, Tables 6 and 7 are given. They show the probable errors of the measures with steel and invar tapes, computed independently.

The results shown in Tables 6 and 7 indicate that the measures with invar tapes are considerably better than those with steel tapes.

If we compare the probable errors obtained before combining with the probable errors due to comparator (as the latter are the same with both kinds of tape), it will be seen that the probable errors from the invar tapes are less than half of those from the steel, if we except the Royalton Base, where the measures with the steel tapes were exceptionally accordant. These probable errors for the measures with steel tapes agree reasonably well with those obtained heretofore, thus showing that a comparison between the measures with invar and steel tapes on these six bases is equivalent to a general comparison between the two metals.

TABLE 6.—PROBABLE ERRORS FOR MEASURES WITH INVAR TAPES.

Base.	Partial.	Due to comparator.	Combined.
	mm.	mm.	mm.
Point Isabel	± 2.83	± 1.48	± 3.19 or 1 part in 2 310 000
Willamette	± 3.13	± 2.80	± 4.20 " 1 " " 3 340 000
Tacoma	± 3.25	± 2.41	± 4.05 " 1 " " 2 980 000
Stephen	± 4.12	± 1.84	± 4.51 " 1 " " 2 040 000
Brown Valley	± 2.06	± 1.64	± 2.64 " 1 " " 3 110 000
Royalton	± 3.40	± 1.93	± 3.91 " 1 " " 2 460 000
			Mean, 1 part in 2 630 000

TABLE 7.—PROBABLE ERRORS FOR MEASURES WITH STEEL TAPES.

Base.	Partial.	Due to comparator.	Combined.
	mm.	mm.	mm.
Point Isabel	± 5.49	± 1.48	± 5.68 or 1 part in 1 300 000
Willamette	± 7.59	± 2.80	± 8.09 " 1 " " 1 730 000
Tacoma	± 6.98	± 2.41	± 7.39 " 1 " " 1 630 000
Stephen	± 7.99	± 1.84	± 8.20 " 1 " " 1 120 000
Brown Valley	± 5.55	± 1.64	± 5.79 " 1 " " 1 420 000
Royalton	± 3.80	± 1.93	± 4.27 " 1 " " 2 260 000
			Mean, 1 part in 1 500 000

According to these probable errors, the measures with the invar tapes should have a weight of 4 to 1 for measures with steel tapes, in combining them for the final length of a base, if only accidental errors are considered; but, as there are probably small systematic errors affecting both kinds of tapes, a weight of only 2 to 1 was used in the combination for the final length of a base.

Table 8 shows the actual discrepancy between the lengths of the

six bases as determined with invar and steel tapes, and also the proportional part each discrepancy is of the whole base.

TABLE 8.

Base.	Invar minus steel.	Proportional discrepancy.
	mm.	
Point Isabel	+ 16.5	Or 1 part in 448 000
Willamette	+ 22.0	" " " " 637 000
Tacoma	+ 40.8	" " " " 295 000
Stephen	— 19.3	" " " " 478 000
Brown Valley	+ 00.2	" " " " 41 000 000
Royalton	+ 19.9	" " " " 484 000
Average		1 part in 527 000

These differences are large when compared with the probable errors of the bases, but when compared with other instances where two forms of apparatus have been used on the measurement of a base they are very gratifying. A discrepancy of 1 in 100 000 or more between the lengths of a base obtained with two forms of apparatus is the rule rather than the exception. On the measures of 1900, the duplex apparatus gave results differing from those of the tapes by 1 in 175 000, on an average, which is three times the average discrepancy between the measures with the invar and steel tapes, and nearly twice the greatest difference.

The fact that the sign of the discrepancy between the measures with the invar and steel tapes is the same on all the bases, except one, seems to indicate a constant difference between the two sets of tapes. This one exception is probably due to the fact that part of the measures with steel tapes were made in the rain, thus causing a result sufficiently abnormal to warrant its rejection from this consideration. If the average discrepancy be considered due to errors of temperature, it would require a constant error of 5° on the measures with invar tapes, or 0.17° on the measures with steel tapes, to account for it. The temperature for the invar tapes may be in error a degree, or even a little more, but they are certainly not in error to the extent of 5 degrees. An error of 0.1°, or more, in determining the temperature of the steel tapes, however, either in the standardization or on the measures, or on both, is not only possible, but probable.

Part of these discrepancies may be due to the wind effect on the

tapes. Owing to the twisting of the invar tapes, they present a little of the flat surface to the wind, thus making them more sensitive to breezes than the steel tapes. A careful inspection of the various measures, with a view to finding some effect of this character, failed to develop anything, as the other errors were large enough to cause these small errors to disappear in the separate sections.

Discussion of Errors.—The results of measurements with tapes and the discussion of their principal errors have been so fully treated in Appendix 8 of the Coast and Geodetic Survey Report for 1892, that it is unnecessary to consider them herein.

The effect of errors due to faulty alignment, both vertically and horizontally, have been eliminated, as previously mentioned, particularly as they are, to some extent, of a compensating character.

The errors caused by faulty tension, although small and partly compensating, are not so completely so as may be desirable. The working balance was usually compared with the two standard balances just before beginning a day's measure and after its completion, particularly if any change were suspected, or if the balance had received unusual handling. It was also frequently suspended by its own hook and its reading noted to see whether the hand had shifted on its pivot, especially after it had received a sudden jerk from any cause.

A change in the working balance of 25 g. (one of the smallest divisions of the dial) was noted occasionally, but no correction was applied to the results for such small errors. They were more or less compensating, and their effect very small. Observations made upon two invar tapes to determine the effect of small changes from the 15-kg. tension gave 0.04 mm. as the effect for a tape length, or 0.8 mm. per km. for a change of 25 g. in tension. The effect upon the steel tapes was considerably less, because they have less mass per unit length and, therefore, less sag under a given tension.

During the work of standardization, the true 15-kg. tension was held very closely, but on the field measures it was impossible to hold it within 25 g. every time. Errors from this source were of the compensating class, however, and their effect on the total length of a base was probably small.

Errors in marking tape lengths are also compensating to a great extent. The graduation mark of the tape was always in the same plane as the copper marking strip on the post, and a tape length was

marked on this copper strip by a line drawn in prolongation of the graduation line. If there were any constant parallax with either observer, it was eliminated in the double measure of the base, as the observers stood on the same side of the posts in all the measures, the two measures always being made in opposite directions.

When preparing the base line, all posts were made solid before leaving them, using braces if necessary to accomplish this purpose. Care was taken to prevent any one from stepping near a post which was holding a measure, hence errors caused by unstable posts were probably negligible, being more or less compensating. No strain was placed on a post, except that necessary to make the mark, which was perpendicular to the base line and not likely to affect the length, especially as the force required was small.

One of the principal sources of error with the steel tapes is the determination of the actual temperature of the tape at the instant of the measure. When the work is progressing rapidly, the tape is moved through the air at a height above the ground two or three times as great as when it is in position for the measure. The time spent in moving from post to post is greater than that necessary to make the measure, hence the tape and thermometers are subject to the temperature of the air at the height of the movement more than at the height of the measure. The temperature near the ground is frequently quite different from that of the air, hence the temperature acquired during the movement is not that of the measure. The tape probably assumes the temperature of its position much more quickly than the thermometers, consequently the thermometers fail to give the actual temperature of the tape at the instant of measure. The fact that the thermometer did not give the temperature of the air in the position of the measure was frequently noticeable when a delay was necessary, as, for example, to measure a set-up. The thermometer would read practically the same while the work was progressing without interruption, but would jump a few tenths of a degree every time a delay occurred, and then return to the original reading as soon as the regular movement was resumed. Errors of this class were more prominent with the steel measures at night than with the invar work in daylight. The temperatures were much more irregular during the day work, of course, but the change during any delay was rarely greater than the usual changes when the work progressed without interruption. In any case,

it was much less than twenty-eight (the ratio of the coefficients of expansion of the two metals) times the temperature differences obtained with the steel tapes.

Cost.—The total cost of the measurement of these six base lines was approximately $7 000, which is at the rate of $116 per km. This includes the cost of standardization, both in the field and at the Bureau of Standards, all expenses of travel and transportation for party and outfit, cost of computations, and preparation of results for publication.

If invar tapes only had been used on these measures, the field standardization would have been unnecessary, thereby reducing the cost very materially, probably to less than $100 per km.

The cost of the preparation of the last three bases was $24 per km. The various operations on the other bases were so interwoven that the cost of the different parts cannot be separated.

Advantages of Invar Tapes.—The principal advantages of invar tapes over steel tapes, developed from this season's work, may be summed up as follows:

Invar tapes may be standardized at the Bureau of Standards instead of in the field, thus effecting a very material saving in both time and money.

The measures with invar tapes may be made in daylight instead of at night, as required with steel tapes, thus increasing the speed and accuracy of the work and further reducing the cost. One objection to the use of the tapes at night is that the moisture from the atmosphere condenses upon the tape and thermometer, increasing the weight of the former and causing the latter to give temperatures which are not as accurate as could be desired. Although errors from this latter source were very noticeable on parts of the measures they were never large enough to warrant a re-measure or a delay of the work.

Owing to the very small coefficient of expansion of the invar tapes, the effect of errors in the determination of the temperatures of the tapes is much less for the invar tapes than for the steel tapes, even though the former are used during the day and the latter at night.

The invar tapes require a little more care in their manipulation than the steel tapes, in order not to bend them to a radius of less than about 8 in. They are a little more sensitive to wind effect than the steel tapes, as the flat surface is frequently twisted considerably from the horizontal plane when in use.

As already stated, steel tapes, when used at night and standardized in the field under conditions similar to those obtained on the measure, give results of the same order of accuracy as bar apparatus, and with about one-third of the cost. The results obtained on these six base lines show that invar tapes give much more accurate results than steel tapes, and are also more economical. Consequently it may be stated that, if the iced-bar apparatus is excepted, the use of which is too expensive for the measurement of base lines, the invar tape is the most economical and accurate apparatus for the measurement of primary bases.*

* A more detailed account of this work will be published in the Coast and Geodetic Survey Report for 1907, which may be obtained about January 1st, 1908, by those interested, on request to the Superintendent of the Coast and Geodetic Survey, Washington, D. C.

DISCUSSION.

J. A. Ockerson, M. Am. Soc. C. E. (by letter).—Mr. French's account of the progress made in the use of steel tapes in geodetic work is interesting and valuable. The old-time methods of using bars or rods for the measurement of base lines were both laborious and expensive, and, as a consequence, the intervals between such lines in a system of triangulation were entirely too long. Mr. Ockerson.

Professor Woodward's paper on long steel tapes, and the discussion thereon,* gave an account of the use of steel tapes in connection with the triangulation work of the Coast Survey.

The official reports of the Mississippi and Missouri River Commissions, of earlier date by several years, gave accounts of the steel-tape work on their respective surveys, which included high-grade triangulation, where the length of the triangle sides and the closure required were such as to compare favorably with so-called primary work.

In August, 1880, the writer made use of a steel tape in the measurement of a base line 6 663 ft. long, opposite Grafton, Ill., in connection with the triangulation in that vicinity, and, although the equipment was deficient in many respects, the results obtained were such as to establish practically the use of the steel tape in the extensive triangulation work which followed, from the mouth of the Ohio to the headwaters of the Mississippi.

The writer believes this to be the first use of the steel tape in refined geodetic work, at least in the United States.

The methods of manipulation in the field were modified and improved, in the interest of both economy and accuracy, as experience developed the defects.

Mr. O. B. Wheeler, of the Missouri River Commission, is entitled to the credit of introducing an accurate tension adjuster, which is described in the Annual Report of that Commission for 1886.

The use of metal strips on which to mark the graduated extremities of the successive tape lengths was also developed in the river surveys. This was a very important step, as it virtually permitted the graphical results of each tape length to be transferred from the field to the office, where the discussion could be taken up at leisure.

Mr. Marshall, in connection with the survey of the Red River, made a number of improvements, among which was the use of two tapes of different metals at one and the same time.

The greatest source of error in the use of the steel tape lies in failure to secure its temperature, as it is much more sensitive to changes than the thermometer used in connection therewith. The writer had in mind a method of diminishing the difference between

* *Transactions*, Am. Soc. C. E., Vol. XXX, pp. 81-107 and 638-652.

Mr. Ockerson.

the two, by the construction of a thermometer with an elongated bulb of the same material as the tape, and perhaps extending the metal along the back of the glass scale tube. The invar tape, apparently, eliminates much of the objection to the steel tape incident to changes of temperature.

In the Mississippi River triangulation, the measurement of base lines became so easy that the general practice called for a base line at intervals of about 12 triangles. That is to say, the instrumental errors of centering both instrument and target, and errors of pointing, were larger than the errors of base measurement, hence such errors were largely localized by the use of frequent bases.

In the writer's opinion, each tape should be standardized by measuring a primary base the length of which has been determined by a Repsold or other refined base-measuring apparatus. The measuring should be done under conditions and by methods identical with those to be used in the measurement of a new base, in preference to relying on a laboratory determination of the length of the tape.

Table 9 gives some results of base-line measurements with steel tapes on the Mississippi River Triangulation. The measurements were generally made in the morning, before sunrise, when changes of temperature were not very rapid. The lengths are given in round numbers, omitting the decimals.

TABLE 9.—Some Results of Base-Line Measurements with Steel Tapes on the Mississippi River Triangulation.

Location.	Length of base line, in feet.	Discrepancy between successive measurements.
New Boston	18 066	1 in 759 000
Rapid City	5 624	" 594 917
East Dubuque	7 105	" 346 965
Cassville	7 001	" 266 412
Prairie du Chien	5 312	" 265 000
De Soto	6 486	" 929 300
Trempeleau	5 223	" 2 396 000
Wabasha	6 783	" 740 000
Red Wing	5 379	" 8 400 000
Fort Snelling	5 400	" 517 000
Monticello	5 401	" 1 475 000
Rice	5 700	" 2 969 000
Brainard	5 400	" 438 400
Aitkin	4 798	" 1 304 000

The method of handling the tape is shown by the photographs on Plate XXIX. Single measurements of lines 1 mile long have been made in 28 min. In the lines cited, no effort was made to secure a very high degree of accuracy, but simply to keep within the limit of discrepancy between two measurements, 1 in 250 000, as prescribed.

FIG. 1.—MEASURING FORT SNELLING BASE LINE WITH 300-FT. STEEL TAPE, REAR END OF TAPE.

FIG. 2.—FRONT END OF TAPE, FORT SNELLING BASE LINE, SHOWING TENSION DEVICE.

HORACE ANDREWS, M. AM. SOC. C. E. (by letter).—The engineering profession is indebted to Mr. French for his clear and useful exposition of the practical adaptability of invar to field use. Mr. Andrew

There would seem to be little left to be desired in base-measuring apparatus, now that temperature corrections are so well eliminated. The history of base measurement has been one of constant struggle against the uncertainty of temperature corrections. At present, the use of iced bars, steel tapes, and night work, enables high precision to be attained, together with speed and economy passing all former experiences. A further and most important advance, from the economical standpoint, is now assured through the use of this wonderful alloy.

Previous to the six base lines referred to by Mr. French, some 35 base lines of primary importance had been measured in the United States. The three earliest, one in 1834 and two in 1844, measured with the Hassler apparatus, had an average probable error of $\frac{1}{214\,000}$. Between 1847 and 1873, seven bases were measured by the Coast Survey, with the Bache-Würdemann apparatus, an average probable error of $\frac{1}{437\,000}$ being indicated. Similar apparatus used by the United States Lake Survey, from 1870 to 1875, gave the average probable error of five bases as $\frac{1}{730\,000}$. The Repsold apparatus then came into use on the Lake Survey, three bases being measured, with an average probable error of only $\frac{1}{1\,075\,000}$. The United States Coast and Geodetic Survey, after 1873, measured eight bases with various apparatus. Two of these, measured in 1891-92 with the iced bar and steel tape, showed excellent results. The average probable error of these eight bases was $\frac{1}{857\,000}$. Then came the phenomenal achievement of 1900, when the United States Coast and Geodetic Survey, having commissioned one field party to measure nine bases in a working season, the aim being to secure a precision of about $\frac{1}{500\,000}$, obtained, not only unprecedented economy of time and money, but an average probable error of only $\frac{1}{1\,150\,000}$. This success was due to the use of the iced bar and steel tapes, as mentioned by Mr. French; the advantages of invar are those pointed out by him, and are irrespective of the higher precision which was incidentally obtained.

Obviously, invar will be an admirable material for precision leveling-rods. Its use for pendulum rods was one of the first suggested. A pendulum, supported by an iron rod, will change its rate about 1

Mr. Andrews. min. a week, if subjected to a change of temperature of 30° fahr., but, with invar having the coefficient given by the author, the change of rate would be only 2 sec.

It would be interesting to know the exact proportions of nickel and steel entering into the composition of the invar tapes. In view of the fact that the coefficient of expansion given in Table 2, is only one-half that found in Guillaume's 36% of nickel alloy, it would seem that some change must have been made in the proportions, and if the name, "invar," is to be adopted, it would be well to have it apply to a definite nickel-steel alloy; at present, there are two "invar" alloys, one of which has only half the invariability of the other.

Engineering measurements in general must be made under all conditions of temperature, and it will be of great advantage to be independent of temperature corrections. The writer has found it very advisable to keep temperature notes for important steel-tape measurements, and correct, under the rule of 0.01 ft. in 100 ft., for each 15° fahr. In the surveys for the Boston Back Bay tract, it has been stated that brass tapes were used, and with the correction of 0.01 ft. in 100 ft. for 10° fahr. change of temperature. With invar, the correction would appear to be 0.001 ft. in 100 ft. for each 44° fahr., so that temperature corrections would in general be negligible.

Mr. Cummings. NOAH CUMMINGS, ASSOC. M. AM. SOC. C. E.—The low coefficient of expansion of invar makes it a most desirable material for a tape, and the results given in the paper show that it has proved very satisfactory for base-line measurements.

It could be used to advantage for measuring base lines for city or bridge triangulations, or wherever great accuracy is required in base-line work. However, there seem to be serious objections to the use of the invar tape for ordinary city surveying, even though its low coefficient of expansion would practically eliminate temperature corrections. It is more easily bent and less elastic than steel, and, according to the makers, requires a reel 16 in. in diameter. Thus great care must be taken in handling it, and it would need to be retested every time someone happened to run into it while a measurement was being made or in case it was stepped on, either of which may easily happen in ordinary street surveying.

It is a question whether surveying needs to be done more accurately than the holding of the points established by the survey. When street corners are marked by stone monuments placed near the surface of the sidewalk, there is a possible movement due to frosts and to excavations for building and street construction purposes. If the invar tape should be developed so as to be more easily handled and then come into use for city surveying, a greater degree of accuracy would be obtained; but, to secure the benefits of this, the points established would have to be marked on stone or concrete monuments either below or extending below the frost line.

Mr. French.

O. B. French, M. Am. Soc. C. E. (by letter).—Mr. Ockerson states that, in his opinion:

"Each tape should be standardized by measuring a primary base the length of which has been determined by a Repsold or other refined base-measuring apparatus."

And that:

"The measuring should be done under conditions and by methods identical with those to be used in the measurement of a new base, in preference to relying on a laboratory determination of the length of the tape."

The writer agrees fully with Mr. Ockerson, when ordinary bars or tapes with coefficients of expansion of ordinary magnitude are used. The Coast and Geodetic Survey has practically done just what he recommends. It is too expensive, of course, to use the iced-bar apparatus for the measurement of a whole base, but the same thing is accomplished when one measures a fraction of the base (or a comparator alongside of it), as was done in both the 1900 and the 1906 work for the standardization of the duplex bars and steel tapes.

It is believed that the errors due to a laboratory standardization are mostly owing to temperature irregularities, or to the difference in temperature effects between the work in the laboratory and that in the field. If this is the case, then one should be able to secure satisfactory laboratory standardization for tapes having small coefficients of expansion, such as those possessed by invar tapes. The results of the measurements in 1906, where all the bases were measured with both steel tapes and invar tapes, the former being standardized in the field under conditions as nearly as possible like those obtained on the base measures, and the latter having only a laboratory standardization, demonstrate, it seems to the writer, as completely as necessary that the laboratory standardization of the invar tapes is certainly as good as the field standardization of the steel tapes, and that when the field work of the two kinds of tapes is compared it must be acknowledged that the results with the invar tapes are superior to those with the steel tapes, notwithstanding the fact that the former were used at any hour during daylight and the latter only at night.

In answer to Mr. Andrews' desire for a definite alloy to be known as "invar," it may be stated that such an alloy, which will have an invariable coefficient of expansion, is probably not practicable at present. A very small change in the proportions of the two metals causes a material change in the coefficient of expansion of the resulting alloy. The fact, too, that one combination has a coefficient which is double that of another is of slight consequence when the smallness of these coefficients is considered. Only on extended measures of very high precision would it be necessary to consider the coefficient of expansion at all, and then, naturally, it should be determined accurately for the particular tape used.

The use of the invar tape whenever it can be cared for easily is certainly preferable if accurate measurements are desired.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1070.

NOTES ON RAINFALL AT SAVANNAH, GEORGIA.

By J. de Bruyn-Kops, M. Am. Soc. C. E.

The writer would not have deemed this paper of sufficient interest to present to the Society were it not for the fact that A. Prescott Folwell, M. Am. Soc. C. E., in a recent book* states that:

"Professor Talbot gives as a formula of maximum rates of rainfall in the eastern part of the country $R = \frac{105}{T + 15}$, which agrees quite closely with Plate IV, South Atlantic States, up to 30 min., but gives too small values for longer periods."

The curve on Plate IV, page 125, of that book, shows "Rates of Rainfall Storms of the Second Class"; and, from the discussion at the bottom of page 56, it is inferred that "Storms of the Second Class" are those that give a certain rate of rainfall approximately five times in ten years, which is equivalent to once in two years. Now, as Savannah is about at the center of the territory named, this curve should apply to that city. The writer, therefore, compared it with some data he had worked out in 1896, while he was Assistant City Engineer of Savannah, and, finding that it did not agree with the results obtained at that time, he obtained from the United States Weather Bureau the data relating to all storms occurring during the years 1889 to 1906, inclusive, as recorded by the automatic rain gauge at Savannah, which gave a precipitation equal to or greater than 0.25 in. in 5 min. or 0.75 in. in 1 hour. These data have been put into the shape here pre-

*"Sewerage," 1907 Edition, Article 17. p. 45.

sented, in the hope that they may be found useful to others engaged in drainage works.

Referring to the quotations given above, it seems to the writer that such expressions as "in the eastern part of the country," and "South Atlantic and Gulf States" are entirely too comprehensive in their scope, and that it will be found that no single formula is even approximately correct for such a large area as that indicated; therefore, in promulgating such formulas, it should be distinctly stated from what data and locality they were deduced, as well as the method of deduction.

The following is the method of treating the data used by the writer: As received from the United States Weather Bureau, the data were in the form of accumulated amounts of precipitation for each 5 min. that the storm lasted; a sample case being that of the storm on December 8th, 1890, which is here reproduced:

Minutes	5	10	15	20	25	30
Accumulated precipitation	0.20	0.32	0.50	0.75	0.95	1.03

Now, it is evident that the natural course of this storm presents rates of precipitation as follows:

Minutes	5	10	15	20	25	30
Accumulated precipitation	0.20	0.32	0.50	0.75	0.95	1.03
Rate, in inches per hour	2.40	1.92	2.00	2.25	2.28	2.06

The greatest rate of precipitation is 2.40 in. per hour for the space of 5 min. It has seemed to the writer, however, that the main consideration is not the rates of fall given by the natural course of the storm, but the maximum rates for the different durations; therefore, the storm data have been rearranged so as to exhibit this maximum, as follows:

Minutes	5	10	15	20	25	30
Accumulated precipitation	0.20	0.32	0.50	0.75	0.95	1.03
Precipitation in each 5 min.	0.20	0.12	0.18	0.25	0.20	0.08
Maximum accumulated precipitation	0.25	0.45	0.63	0.75	0.95	1.03
Maximum rate of precipitation	3.00	2.70	2.52	2.25	2.28	2.06

The line marked "Precipitation in each 5 min." is obtained by putting in the 5-min. column the given accumulation for that period, then subtracting the given 5-min. accumulation from the given 10-min. accumulation and putting the result in the 10-min. column, and so on. The amounts in the line marked "Maximum accumulated precipitation" are: for 5 min., the amount precipitated during the period between 15 and 20 min.; for 10 min., the precipitation during the period between 15 and 25 min.; for 15 min., the precipitation during the period between 10 and 25 min.; and so on, selecting in each case the period, in any part of the storm, that gives the maximum precipitation for that period. In this manner, 162 storms were examined, being all that were reported by the United States Weather Bureau, for the years, and of the intensity, given. Table 1 shows the result of this investigation.

Two other points should be explained, in reference to the table: First, in tabulating the results of any storm which gave a greater rate of precipitation for a longer period than for a shorter one, the greater rate was tabulated for the shorter as well as for the longer period. For instance, in the storm of December 8th, 1890, it will be seen that the maximum rate for 20 min. was 2.25 in., while the maximum rate for 25 min. was 2.28 in. The rate has been tabulated as 2.28 in. in both cases, on the general grounds that if it rained at the rate of 2.28 in. for 25 min., it is reasonable to say that it rained at an equal rate for a less time.

The other point is, that all storms have been extended, from the point of their ending to the limit of time to which the investigation was carried (100 min.). This extension was made on the ground that if a storm gave, say, 0.5 in. of precipitation in 10 min. and then stopped, the effect on a drainage area in which the time of accumulation at the outlet was more than 10 min. would certainly be not less than if the same storm had precipitated 0.5 in. in the accumulation time of the area, whether it were 15 or 100 min.; and the probability is that it would be greater. In Table 1, therefore, the extended rates have been calculated on the total accumulated amounts as though they had been precipitated in the extended times. In order that these extended rates may be distinguished from the actual rates, they have been printed in italics.

On Plate XXX have been plotted about one hundred of the highest

values for each 5-min. period, as taken from Table 1. At the maximum value, and from there counting downward 9, 18, 36, 54, 72, and 90, the plottings have been made larger than the others. As the number of years examined is 18, these large plottings indicate the rates of maximum storms and those occurring on an average of once in two years, and one, two, three, four, and five times each year. Through these points have been passed, with more or less success, regular curves, the equations for which are noted at the right of Plate XXX.

The following comparison of Professor Talbot's "Maximum curve for the eastern part of the country," with the maximum curve for Savannah, Ga., as deduced by the writer, is of interest:

Minutes	5	10	15	20	25	30
Talbot $R = \frac{105}{T+15}$	5.25	4.20	3.50	3.00	2.63	2.34
Savannah $R = \frac{191}{T+19}$	7.96	6.59	5.62	4.90	4.34	3.90

Minutes	35	40	45	50	55	60	80	100
Talbot	2.10	1.91	1.75	1.62	1.50	1.40	1.10	0.92
Savannah	3.54	3.24	2.99	2.77	2.59	2.42	1.93	1.60

The comparison of Folwell's curve for "Storms of the second class for the South Atlantic and Gulf States," with the corresponding curve for storms occurring once in two years, as found for Savannah conditions, is also made, so that the differences can be readily seen:

Minutes	5	10	15	20	25	30	35
Folwell	5.50	4.25	3.54	3.05	2.68	2.44	2.26
Savannah $R = \frac{163}{T+27}$	5.10	4.41	3.88	3.46	3.14	2.86	2.63

Minutes	40	45	50	55	60
Folwell	2.11	2.00	1.92	1.86	1.80
Savannah	2.44	2.26	2.12	1.99	1.87

As Folwell gives no formula for his curve, the evaluation was made from the curve shown on page 125 of the book previously quoted.

No. of Storm.	Date.	Duration of Periods, in Minutes.													
		5	10	15	20	25	30	35	40	45	50	55	60	80	100
25	July 18, 1891.	5.40	5.10	5.00	4.50	4.08	3.64	3.18	*2.78*	*2.46*	*2.22*	*2.02*	*1.85*	*1.39*	*1.11*
26	July 25, 1891.	3.24	2.58	*1.72*	*1.29*	*1.03*	*0.86*	*0.73*	*0.65*	*0.57*	*0.52*	*0.47*	*0.43*	*0.32*	*0.26*
27	July 26, 1891.	3.00	2.52	2.40	2.28	2.04	1.91	1.91	1.77	1.62	1.57	1.53	1.45	*1.08*	*0.87*
28	July 28, 1891.	3.00	2.88	2.64	2.25	2.23	2.02	1.94	1.74	*1.54*	*1.39*	*1.26*	*1.16*	*0.87*	*0.70*
29	Aug. 2, 1891.	2.40	1.86	1.52	1.44	*1.15*	*0.96*	*0.82*	*0.72*	*0.64*	*0.58*	*0.52*	*0.48*	*0.36*	*0.29*
30	Aug. 6, 1891.	3.00	2.40	1.88	1.53	*1.22*	*1.02*	*0.87*	*0.77*	*0.68*	*0.61*	*0.56*	*0.51*	*0.38*	*0.31*
31	Aug. 13, 1891.	3.84	2.82	2.48	2.31	2.16	1.94	1.74	1.58	1.50	1.48	1.38	1.30	*0.98*	*0.78*
32	Aug. 22, 1891.	3.60	3.20	3.20	3.00	2.59	2.40	2.11	1.91	1.71	1.57	*1.43*	*1.31*	*0.98*	*0.79*
33	Aug. 28, 1891.	6.00	4.50	4.00	3.60	3.24	2.90	2.53	2.24	2.01	1.84	1.69	1.60	1.60	1.51
34	July 18, 1892.	3.36	3.00	2.80	2.31	1.95	1.95	1.95	1.95	1.77	*1.60*	*1.45*	*1.33*	*1.00*	*0.80*
35	Aug. 10, 1892.	3.96	2.48	2.48	2.10	*1.68*	*1.40*	*1.20*	*1.05*	*0.93*	*0.84*	*0.76*	*0.70*	*0.52*	*0.42*
36	Sept. 12, 1892.	3.00	2.40	2.00	1.71	1.44	1.26	1.12	1.02	0.93	0.90	*0.82*	*0.75*	*0.56*	*0.45*
37	Sept. 19, 1892.	2.40	2.10	1.80	*1.35*	*1.08*	*0.90*	*0.77*	*0.68*	*0.60*	*0.54*	*0.49*	*0.45*	*0.34*	*0.27*
38	Sept. 20, 1892.	1.80	1.72	1.68	1.65	1.51	1.40	1.32	1.25	1.16	1.07	0.97	0.93	0.81	0.75
39	June 7, 1893.	3.12	2.46	2.24	2.10	1.87	1.62	*1.39*	*1.22*	*1.08*	*0.97*	*0.88*	*0.81*	*0.61*	*0.49*
40	July 15, 1893.	2.40	2.28	2.16	2.07	1.92	1.72	1.60	1.53	1.40	1.30	1.20	1.15	1.07	*0.90*
41	Aug. 3, 1893.	3.00	2.70	2.28	2.01	1.68	*1.40*	*1.20*	*1.05*	*0.93*	*0.84*	*0.76*	*0.70*	*0.53*	*0.42*
42	Aug. 14, 1893.	3.00	3.00	2.44	*1.83*	*1.46*	*1.22*	*1.05*	*0.92*	*0.81*	*0.73*	*0.66*	*0.61*	*0.46*	*0.37*
43	Aug. 20, 1893.	3.48	3.24	2.40	*1.80*	*1.44*	*1.20*	*1.03*	*0.90*	*0.80*	*0.72*	*0.65*	*0.60*	*0.45*	*0.36*
44	Aug. 27, 1893.	2.64	2.52	2.20	2.04	1.92	*1.60*	*1.38*	*1.20*	*1.06*	*0.96*	*0.87*	*0.80*	*0.60*	*0.48*
45	Sept. 9, 1893.	3.36	3.00	2.85	2.85	2.66	2.52	2.34	2.27	2.26	2.22	2.18	2.15	1.83	1.57
46	Sept. 13, 1893.	2.40	2.28	1.92	1.86	1.73	1.62	1.53	1.41	1.33	1.22	1.13	1.05	0.88	*0.67*
47	Oct. 3, 1893.	3.60	3.60	2.88	2.34	2.24	2.24	2.03	1.86	1.82	1.72	1.59	1.49	1.33	*1.24*
48	Nov. 27, 1893.	3.00	2.40	2.00	*1.50*	*1.20*	*1.00*	*0.86*	*0.75*	*0.67*	*0.60*	*0.55*	*0.50*	*0.38*	*0.30*
49	June 23, 1894.	3.00	2.70	2.60	2.25	1.87	1.62	1.44	*1.26*	*1.12*	*1.00*	*0.92*	*0.84*	*0.63*	*0.50*
50	July 3, 1894.	2.64	2.10	1.44	*1.08*	*0.86*	*0.72*	*0.62*	*0.54*	*0.48*	*0.43*	*0.39*	*0.36*	*0.27*	*0.22*
51	July 18, 1894.	6.00	5.40	4.80	4.20	3.72	3.20	3.01	2.70	2.46	2.28	*2.07*	*1.90*	*1.43*	*1.14*

TABLE 1.—RATE OF RAINFALL AT SAVANNAH, GEORGIA (IN INCHES PER HOUR), AS RECORDED BY THE AUTOMATIC RAIN GAUGE OF THE U. S. WEATHER BUREAU, FOR ALL STORMS IN WHICH THE PRECIPITATION AMOUNTED TO 0.25 IN. IN ANY 5 MIN., OR 0.75 IN. IN 1 HOUR, DURING THE YEARS 1889 TO 1906, INCLUSIVE,

NOTE: The figures in italics show the extended rates, after the storm had stopped.

No. of Storm.	Date.	Duration of Periods, in Minutes.													
		5	10	15	20	25	30	35	40	45	50	55	60	80	100
1	June 27, 1889.	3.96	3.84	2.84	2.43	2.09	1.92	1.79	1.70	1.60	1.55	*1.41*	*1.29*	*0.96*	*0.77*
2	June 28, 1889.	2.28	2.04	1.76	1.62	1.44	1.30	1.29	1.22	1.14	*1.03*	*0.93*	*0.86*	*0.65*	*0.52*
3	July 6, 1889.	3.60	2.34	1.88	1.68	1.56	1.38	*1.18*	*1.04*	*0.91*	*0.83*	*0.75*	*0.69*	*0.52*	*0.41*
4	Aug. 6, 1889.	4.56	4.38	4.32	4.17	3.91	3.66	3.34	3.08	2.83	2.59	2.38	2.22	1.71	1.44
5	Aug. 8, 1889.	4.80	4.80	3.80	3.15	2.59	*2.16*	*1.85*	*1.62*	*1.43*	*1.30*	*1.17*	*1.08*	*0.81*	*0.65*
6	Sept. 1, 1889.	4.20	3.90	3.80	*2.85*	*2.28*	*1.90*	*1.63*	*1.43*	*1.26*	*1.14*	*1.03*	*0.95*	*0.71*	*0.57*
7	Sept. 6, 1889.	3.60	3.30	2.80	2.52	2.14	1.90	1.72	*1.50*	*1.33*	*1.20*	*1.09*	*1.00*	*0.75*	*0.60*
8	May 3, 1890.	3.00	2.22	2.20	*1.65*	*1.32*	*1.10*	*0.94*	*0.83*	*0.73*	*0.66*	*0.60*	*0.55*	*0.41*	*0.33*
9	June 11, 1890.	3.60	3.00	2.60	2.25	1.92	1.70	1.55	*1.35*	*1.20*	*1.08*	*0.98*	*0.90*	*0.68*	*0.54*
10	June 20, 1890.	2.40	1.74	1.60	1.71	1.49	*1.24*	*1.06*	*0.93*	*0.82*	*0.74*	*0.67*	*0.62*	*0.47*	*0.37*
11	July 1, 1890.	3.00	3.00	2.40	*1.80*	*1.44*	*1.20*	*1.03*	*0.90*	*0.80*	*0.72*	*0.65*	*0.60*	*0.45*	*0.36*
12	July 8, 1890.	4.20	4.20	3.40	2.88	*2.30*	*1.92*	*1.65*	*1.44*	*1.28*	*1.15*	*1.05*	*0.96*	*0.72*	*0.58*
13	July 27, 1890.	3.36	2.88	2.28	1.86	1.56	1.40	1.24	1.11	1.00	0.91	0.91	0.91	0.91	0.79
14	Sept. 1-2, 1890.	2.40	1.62	1.36	1.20	1.13	1.00	0.93	0.93	0.88	0.82	0.77	0.73	0.71	0.63
15	Sept. 2, 1890.	3.00	2.70	2.20	1.86	1.66	1.54	1.44	1.32	1.21	1.13	1.05	0.99	0.91	0.87
16	Sept. 3, 1890.	2.28	1.44	1.08	0.87	0.79	0.77	0.77	*0.67*	*0.60*	*0.54*	*0.49*	*0.45*	*0.34*	*0.27*
17	Sept. 11, 1890.	3.00	2.04	1.64	1.32	*1.06*	*0.88*	*0.76*	*0.66*	*0.59*	*0.53*	*0.48*	*0.44*	*0.33*	*0.26*
18	Sept. 13, 1890.	2.40	2.40	2.00	1.95	*1.56*	*1.30*	*1.11*	*0.98*	*0.86*	*0.78*	*0.71*	*0.65*	*0.49*	*0.39*
19	Sept. 15, 1890.	1.80	1.80	1.56	1.38	1.22	1.10	1.00	0.92	0.85	*0.77*	*0.70*	*0.64*	*0.48*	*0.38*
20	Sept. 22, 1890.	2.40	1.80	1.24	*0.93*	*0.74*	*0.62*	*0.53*	*0.47*	*0.41*	*0.37*	*0.34*	*0.31*	*0.23*	*0.19*
21	Sept. 24, 1890.	2.64	2.40	2.28	2.31	2.16	1.94	1.93	1.93	1.93	1.82	1.61	1.40	1.28	*1.02*
22	Oct. 22, 1890.	4.20	3.60	3.12	2.76	2.38	2.02	*1.73*	*1.52*	*1.34*	*1.21*	*1.10*	*1.01*	*0.76*	*0.61*
23	Dec. 8, 1890.	3.00	2.70	2.52	2.28	2.28	2.06	*1.77*	*1.56*	*1.37*	*1.24*	*1.12*	*1.03*	*0.77*	*0.62*
24	May 27, 1891.	4.20	3.00	2.12	1.68	1.42	*1.18*	*1.01*	*0.88*	*0.78*	*0.71*	*0.64*	*0.59*	*0.44*	*0.35*

No. of Storm.	Date.	Duration of Periods, in Minutes.													
		5	10	15	20	25	30	35	40	45	50	55	60	80	100
79	July 11, 1897.	7.80	6.48	5.28	4.29	3.48	*2.90*	*2.49*	*2.17*	*1.93*	*1.74*	*1.58*	*1.45*	*1.09*	*0.87*
80	July 22, 1897.	3.36	3.00	2.80	2.70	2.52	2.34	2.25	2.25	2.18	2.14	1.99	1.84	*1.38*	*1.10*
81	Aug. 14, 1897.	4.56	4.14	3.92	3.81	3.48	3.16	2.89	2.58	2.33	2.14	1.97	1.83	1.39	*1.12*
82	Aug. 15, 1897.	4.32	3.36	2.48	*1.86*	*1.49*	*1.24*	*1.06*	*0.93*	*0.82*	*0.74*	*0.68*	*0.62*	*0.47*	*0.37*
83	June 13, 1898.	2.76	2.46	2.12	1.83	1.63	1.48	1.32	*1.15*	*1.02*	*0.92*	*0.84*	*0.77*	*0.58*	*0.46*
84	June 19, 1898.	3.24	3.18	3.16	2.73	2.35	2.26	2.03	*1.77*	*1.56*	*1.42*	*1.28*	*1.18*	*0.89*	*0.71*
85	July 5–6, 1898.	2.64	2.34	1.96	1.74	1.46	*1.22*	*1.04*	*0.92*	*0.81*	*0.73*	*0.66*	*0.61*	*0.46*	*0.37*
86	July 12–13, 1898.	2.64	2.22	2.04	1.92	1.58	1.36	1.34	*1.17*	*1.03*	*0.94*	*0.85*	*0.78*	*0.59*	*0.47*
87	Aug. 1, 1898.	2.16	1.80	1.76	1.56	1.39	1.34	1.26	*1.10*	*0.97*	*0.88*	*0.80*	*0.73*	*0.55*	*0.44*
88	Aug. 4, 1898.	3.48	2.34	2.00	1.92	1.78	*1.48*	*1.27*	*1.11*	*0.98*	*0.89*	*0.78*	*0.74*	*0.56*	*0.44*
89	Aug. 16–17, 1898.	2.40	2.04	1.64	1.56	*1.25*	*1.04*	*0.89*	*0.78*	*0.69*	*0.62*	*0.57*	*0.52*	*0.39*	*0.31*
90	Aug. 16–17, 1898.	1.92	1.62	1.68	1.53	1.39	1.28	1.27	1.27	1.27	1.26	1.22	1.18	*0.89*	*0.71*
91	Sept. 4, 1898.	4.32	3.68	3.68	3.39	3.07	2.98	2.83	2.66	2.43	2.20	2.00	*1.84*	*1.38*	*1.10*
92	Sept. 8, 1898.	2.40	1.98	1.85	1.85	1.85	1.74	1.67	1.56	1.48	1.37	1.26	1.18	*0.89*	*0.71*
93	Mar. 28, 1899.	4.44	3.06	2.24	1.74	1.42	1.24	1.20	1.12	1.08	1.03	0.99	0.96	*0.72*	*0.58*
94	July 8–9, 1899.	2.64	2.10	2.00	1.42	1.42	1.28	1.20	1.17	1.10	1.08	1.02	0.99	*0.74*	*0.59*
95	Aug. 27–29, 1899.	2.52	2.28	2.08	1.83	1.82	1.70	1.58	1.47	1.38	1.32	1.25	1.19	*0.89*	*0.71*
96	Sept. 17, 1899.	2.28	1.80	1.48	1.41	1.39	1.18	1.15	1.02	1.02	0.95	*0.86*	*0.79*	*0.59*	*0.47*
97	May 18, 1900.	3.36	2.28	1.80	1.56	1.32	1.18	1.07	1.05	0.98	0.92	0.85	0.79	*0.59*	*0.47*
98	May 23, 1900.	1.92	1.68	1.52	1.20	1.04	1.04	1.04	1.04	1.04	0.95	0.87	0.81	*0.61*	*0.49*
99	June 24, 1900.	3.60	3.00	2.76	2.46	2.33	2.08	1.96	1.76	1.57	1.43	*1.29*	*1.19*	*0.89*	*0.71*
100	July 29, 1900.	2.52	2.04	1.64	1.38	1.18	*0.98*	*0.84*	*0.74*	*0.65*	*0.59*	*0.53*	*0.49*	*0.37*	*0.29*
101	Aug. 31, 1900.	2.76	2.52	2.00	1.71	1.49	1.30	1.20	1.10	1.01	*0.91*	*0.83*	*0.76*	*0.57*	*0.46*
102	Sept. 1, 1900.	2.46	2.04	1.80	1.53	1.34	1.20	1.10	*0.96*	*0.85*	*0.77*	*0.70*	*0.64*	*0.48*	*0.38*
103	Sept. 13, 1900.	3.72	3.06	2.68	2.19	*1.75*	*1.46*	*1.26*	*1.09*	*0.97*	*0.88*	*0.80*	*0.73*	*0.55*	*0.44*
104	Oct. 2–3, 1900.	2.64	2.52	2.16	1.71	1.39	1.18	1.16	1.16	1.16	1.16	1.16	1.16	0.96	0.95
105	Nov. 3, 1900.	3.12	2.60	2.60	2.49	2.16	1.92	1.76	1.76	1.60	1.55	1.43	1.34	1.16	*0.93*

TABLE 1.—(*Continued.*)

No. of Storm.	Date.	Duration of Periods, in Minutes.													
		5	10	15	20	25	30	35	40	45	50	55	60	80	100
52	July 31 1894.	3.60	2.40	2.08	1.71	1.56	1.38	1.27	1.20	1.13	*1.02*	*0.93*	*0.85*	*0.64*	*0.51*
53	Aug. 17, 1894.	3.84	3.72	3.60	3.18	2.88	2.58	2.36	2.15	2.00	*1.80*	*1.64*	*1.50*	*1.13*	*0.90*
54	Mar. 12, 1895.	2.64	1.62	*1.08*	*0.81*	*0.64*	*0.54*	*0.46*	*0.41*	*0.35*	*0.32*	*0.29*	*0.27*	*0.20*	*0.16*
55	Apr. 24, 1895.	2.76	1.89	1.60	1.38	1.22	1.08	1.04	1.00	1.00	1.00	1.00	1.00	*0.75*	*0.60*
56	May 24, 1895.	2.40	1.90	1.68	1.38	*1.10*	*0.92*	*0.79*	*0.69*	*0.61*	*0.55*	*0.50*	*0.46*	*0.34*	*0.28*
57	June 13, 1895.	3.36	3.00	2.68	2.46	2.11	*1.76*	*1.51*	*1.32*	*1.17*	*1.06*	*0.96*	*0.88*	*0.66*	*0.53*
58	June 16, 1895.	2.40	2.10	2.00	1.98	1.92	1.80	1.73	1.73	1.73	1.63	1.54	1.43	1.25	1.07
59	June 24, 1895.	2.64	2.22	2.00	2.01	1.73	*1.44*	*1.24*	*1.08*	*0.96*	*0.86*	*0.78*	*0.72*	*0.54*	*0.43*
60	June 30, 1895.	2.52	2.10	*1.40*	*1.05*	*0.84*	*0.70*	*0.60*	*0.53*	*0.47*	*0.42*	*0.38*	*0.35*	*0.26*	*0.21*
61	July 7, 1895.	2.76	1.80	1.60	1.35	1.13	*0.94*	*0.80*	*0.71*	*0.63*	*0.56*	*0.51*	*0.47*	*0.35*	*0.28*
62	July 12, 1895.	2.64	1.74	1.36	1.32	1.13	*0.94*	*0.80*	*0.71*	*0.63*	*0.56*	*0.51*	*0.47*	*0.35*	*0.28*
63	July 13, 1895.	6.60	6.00	5.60	5.10	4.32	3.64	3.18	3.02	2.87	2.68	2.43	2.28	1.71	1.37
64	July 25, 1895.	2.88	2.58	2.48	2.46	2.42	2.26	2.11	1.89	1.69	1.54	1.41	1.35	1.25	*1.12*
65	Aug. 3, 1895.	4.20	3.72	3.20	2.88	2.59	2.34	2.24	2.22	2.13	2.06	1.98	1.87	1.56	1.52
66	Aug. 4, 1895.	3.00	2.76	1.92	1.50	*1.20*	*1.00*	*0.86*	*0.75*	*0.67*	*0.60*	*0.55*	*0.50*	*0.38*	*0.30*
67	Aug. 17, 1895.	2.40	1.74	1.28	1.05	*0.84*	*0.70*	*0.60*	*0.53*	*0.47*	*0.42*	*0.38*	*0.35*	*0.26*	*0.21*
68	Aug. 22, 1895.	3.24	2.40	2.20	2.04	1.92	1.64	1.44	1.28	1.14	1.13	1.13	1.13	*0.99*	*0.79*
69	Aug. 22, 1895.	3.12	2.28	2.00	2.04	1.92	1.62	*1.39*	*1.32*	*1.07*	*0.97*	*0.88*	*0.81*	*0.61*	*0.49*
70	Apr. 24, 1896.	3.00	2.82	2.40	2.01	1.75	1.56	1.42	1.31	1.18	1.12	1.07	1.02	0.82	0.75
71	July 7, 1896.	4.20	3.60	3.40	3.00	2.52	2.16	1.89	1.67	1.49	1.36	1.24	1.15	0.98	*0.78*
72	Aug. 18, 1896.	3.00	3.00	2.85	2.85	2.40	*2.00*	*1.72*	*1.50*	*1.33*	*1.20*	*1.09*	*1.00*	*0.75*	*0.60*
73	Aug. 26, 1896.	4.20	3.78	3.48	3.21	2.86	2.60	2.44	2.24	2.02	1.88	1.74	1.63	*1.22*	*0.98*
74	Feb. 1, 1897.	3.00	1.98	1.52	1.26	1.13	1.08	1.00	0.93	0.84	0.82	0.78	0.76	0.68	0.59
75	Mar. 13, 1897.	4.56	2.64	2.04	1.74	*1.39*	*1.16*	*0.99*	*0.87*	*0.77*	*0.70*	*0.63*	*0.58*	*0.43*	*0.35*
76	Apr. 26, 1897.	3.12	*1.56*	*1.04*	*0.78*	*0.62*	*0.52*	*0.45*	*0.39*	*0.34*	*0.31*	*0.28*	*0.26*	*0.20*	*0.16*
77	June 15, 1897.	3.84	3.36	3.20	2.88	2.66	2.42	2.30	2.13	1.98	1.86	1.72	1.60	*1.20*	*0.96*
78	July 5, 1897.	1.92	1.74	1.68	1.68	1.51	1.48	1.55	1.50	*1.33*	*1.20*	*1.09*	*1.00*	*0.75*	*0.60*

No. of Storm.	Date.	Duration of Periods, in Minutes.													
		5	10	15	20	25	30	35	40	45	50	55	60	80	100
133	July 24, 1904.	3.00	2.88	2.84	2.76	2.78	2.50	2.33	2.16	2.12	2.12	2.04	1.99	1.73	1.39
134	July 28-29, 1904.	3.72	3.36	3.16	2.37	1.90	1.58	1.36	1.18	1.05	0.95	0.86	0.79	0.59	0.47
135	July 28-29, 1904.	2.88	2.46	2.16	1.98	1.80	1.80	1.80	1.80	1.66	1.58	1.44	1.32	0.99	0.79
136	July 28-29, 1904.	4.80	3.96	2.88	2.88	2.71	2.44	2.20	1.92	1.70	1.54	1.39	1.28	0.96	0.77
137	July 28-29, 1904.	3.60	3.54	3.44	3.24	3.00	2.80	2.59	2.29	2.04	1.85	1.69	1.55	1.16	0.93
138	Aug. 3, 1904.	4.32	3.78	3.08	2.31	1.85	1.54	1.32	1.16	1.02	0.92	0.84	0.77	0.58	0.46
139	Aug. 22, 1904.	2.76	2.70	2.48	2.16	2.02	1.86	1.73	1.56	1.38	1.25	1.13	1.04	0.78	0.62
140	Sept. 3, 1904.	2.16	1.98	1.92	1.92	1.82	1.60	1.42	1.32	1.17	1.06	0.95	0.88	0.66	0.53
141	May 7, 1905.	4.20	3.48	2.72	2.64	2.64	2.58	2.39	2.25	2.14	2.04	1.95	1.85	1.39	1.11
142	May 21, 1905.	2.76	1.80	1.20	0.90	0.72	0.60	0.52	0.45	0.40	0.36	0.33	0.30	0.23	0.18
143	July 5, 1905.	3.12	3.00	2.44	1.98	1.73	1.44	1.23	1.08	0.95	0.86	0.78	0.72	0.54	0.43
144	July 6, 1905.	2.04	1.92	1.28	0.96	0.77	0.64	0.55	0.48	0.43	0.38	0.35	0.32	0.24	0.19
145	July 23, 1905.	3.72	1.98	1.32	0.99	0.79	0.66	0.57	0.49	0.44	0.40	0.36	0.33	0.25	0.20
146	July 25, 1905.	2.64	2.22	2.04	1.89	1.80	1.50	1.29	1.13	1.00	0.90	0.82	0.75	0.56	0.45
147	July 28, 1905.	3.96	3.40	3.40	3.39	3.22	2.82	2.68	2.34	2.07	1.87	1.70	1.56	1.17	0.94
148	Aug. 12, 1905.	4.92	3.18	2.12	1.59	1.27	1.06	0.91	0.80	0.70	0.64	0.58	0.53	0.39	0.31
149	Aug. 18, 1905.	4.68	3.90	2.60	1.95	1.56	1.30	1.11	0.97	0.86	0.78	0.71	0.65	0.49	0.39
150	Sept. 13, 1905.	3.00	2.70	2.68	2.61	2.40	2.00	1.72	1.50	1.33	1.20	1.09	1.00	0.75	0.60
151	Sept. 21, 1905.	2.88	2.76	2.72	2.58	2.50	2.18	1.96	1.71	1.51	1.37	1.24	1.14	0.86	0.68
152	Oct. 4, 1905.	3.36	2.94	2.68	2.16	1.87	1.66	1.42	1.25	1.10	1.00	0.90	0.83	0.62	0.50
153	Oct. 26, 1905.	1.80	1.68	1.48	1.38	1.32	1.22	1.04	0.91	0.81	0.73	0.66	0.61	0.46	0.37
154	Apr. 14, 1906.	4.32	3.48	2.68	2.49	2.18	1.82	1.56	1.37	1.21	1.09	0.99	0.91	0.68	0.55
155	June 27, 1906.	2.16	1.98	1.40	1.05	0.84	0.70	0.60	0.53	0.47	0.42	0.38	0.35	0.26	0.21
156	July 30, 1906.	1.80	1.62	1.40	1.05	0.84	0.70	0.60	0.53	0.47	0.42	0.38	0.35	0.26	0.21
157	July 30, 1906.	3.24	2.88	2.44	1.95	1.56	1.30	1.11	0.97	0.86	0.78	0.71	0.65	0.49	0.39
158	Aug. 23, 1906.	2.52	2.16	1.64	1.23	0.98	0.82	0.71	0.62	0.55	0.49	0.45	0.41	0.31	0.24
159	Aug. 27, 1906.	2.64	2.46	2.04	1.98	1.85	1.66	1.42	1.24	1.10	1.00	0.90	0.83	0.62	0.50
160	Aug. 28, 1906.	3.12	2.58	2.28	2.16	1.78	1.48	1.27	1.11	0.98	0.89	0.81	0.74	0.56	0.44
161	Sept. 5, 1906.	2.64	1.62	1.36	1.28	1.42	1.26	1.08	0.95	0.84	0.76	0.69	0.63	0.47	0.38
162	Sept. 26, 1906.	2.64	1.80	1.44	1.08	0.86	0.72	0.62	0.54	0.48	0.43	0.39	0.36	0.27	0.21

TABLE 1.—(*Continued.*)

No. of Storm.	Date.	Duration of Periods, in Minutes.													
		5	10	15	20	25	30	35	40	45	50	55	60	80	100
106	June 17, 1901.	3.60	3.44	3.42	2.76	2.21	1.90	1.86	1.71	1.62	1.55	1.47	1.40	*1.05*	*0.84*
107	July 8, 1901.	3.96	3.18	2.36	2.01	1.92	1.70	1.53	1.38	*1.22*	*1.10*	*1.00*	*0.92*	*0.69*	*0.55*
108	Aug. 7, 1901.	2.40	2.36	2.36	2.19	1.85	*1.54*	*1.32*	*1.15*	*1.02*	*0.92*	*0.84*	*0.77*	*0.58*	*0.46*
109	Aug. 25, 1901.	4.56	3.84	3.24	2.88	2.33	*1.94*	*1.66*	*1.45*	*1.29*	*1.16*	*1.05*	*0.97*	*0.73*	*0.58*
110	Apl. 30, 1902.	1.80	1.68	1.60	1.44	1.44	1.36	1.36	1.36	1.24	1.15	1.11	1.07	0.95	*0.76*
111	May 19, 1902.	2.28	2.16	2.04	1.65	1.34	1.14	1.00	0.90	0.86	0.86	*0.78*	*0.72*	*0.54*	*0.43*
112	July 11–12, 1902.	5.88	4.02	3.96	3.66	3.29	3.19	3.19	3.02	2.85	2.71	2.58	2.42	1.88	*1.51*
113	July 13, 1902.	3.36	3.18	2.94	2.94	2.71	2.46	2.36	2.15	1.92	1.82	1.71	1.61	1.32	*1.06*
114	July 21, 1902.	3.48	2.88	2.12	1.86	1.61	1.36	*1.16*	*1.02*	*0.90*	*0.82*	*0.74*	*0.68*	*0.51*	*0.41*
115	July 27, 1902.	1.80	1.48	1.48	1.38	1.30	1.18	1.07	0.95	*0.82*	*0.74*	*0.68*	*0.63*	*0.47*	*0.37*
116	Aug. 11, 1902.	2.76	2.28	2.04	1.89	1.70	1.64	1.53	1.52	1.43	1.42	1.36	1.30	1.19	*1.05*
117	Aug. 28, 1902.	1.92	1.86	1.68	1.50	1.39	1.34	1.31	1.26	1.25	1.25	1.25	1.24	1.10	*0.88*
118	Sept. 9, 1902.	2.88	2.76	2.64	2.40	2.04	1.76	1.55	*1.35*	*1.20*	*1.08*	*0.98*	*0.90*	*0.68*	*0.54*
119	Dec. 3–4, 1902.	3.84	2.58	1.96	1.86	1.73	1.70	1.62	1.58	1.46	*1.32*	*1.20*	*1.10*	*0.83*	*0.66*
120	June 4, 1903.	3.12	2.76	2.40	2.01	1.63	*1.36*	*1.16*	*1.02*	*0.90*	*0.82*	*0.74*	*0.68*	*0.51*	*0.41*
121	June 8, 1903.	3.00	2.52	2.16	2.01	1.75	*1.46*	*1.25*	*1.09*	*0.97*	*0.88*	*0.80*	*0.73*	*0.55*	*0.44*
122	June 11, 1903.	2.64	2.28	2.00	1.86	1.58	1.52	1.39	1.25	*1.10*	*1.00*	*0.90*	*0.83*	*0.62*	*0.50*
123	June 18, 1903.	2.40	2.10	1.64	1.32	1.20	1.20	1.10	1.01	*0.89*	*0.80*	*0.73*	*0.67*	*0.50*	*0.40*
124	July 9, 1903.	3.72	3.66	3.40	3.30	3.00	2.78	2.63	2.43	2.24	2.15	2.00	1.91	1.58	1.46
125	July 22, 1903.	2.16	1.98	1.96	1.80	1.61	1.40	*1.20*	*1.05*	*0.93*	*0.84*	*0.76*	*0.70*	*0.53*	*0.42*
126	Aug. 15, 1903.	4.44	3.12	3.00	2.28	1.85	1.68	1.63	1.63	1.63	1.63	1.49	1.49	*1.26*	*1.01*
127	Aug. 31, 1903.	3.60	2.40	1.80	1.42	1.42	1.42	1.25	*1.09*	*0.97*	*0.87*	*0.80*	*0.73*	*0.55*	*0.44*
128	Sept. 2, 1903.	6.72	5.64	4.80	4.32	3.91	3.56	3.19	2.80	*2.48*	*2.24*	*2.03*	*1.87*	*1.40*	*1.12*
129	Oct. 16–17, 1903.	2.76	2.16	2.00	2.07	1.85	1.78	*1.53*	*1.33*	*1.18*	*1.07*	*0.97*	*0.89*	*0.66*	*0.53*
130	Nov. 4, 1903.	2.76	2.58	2.16	1.68	1.39	*1.16*	*0.99*	*0.87*	*0.77*	*0.70*	*0.63*	*0.58*	*0.43*	*0.35*
131	June 10, 1904.	2.52	2.34	2.16	1.95	1.82	1.64	1.46	*1.26*	*1.13*	*1.02*	*0.93*	*0.85*	*0.63*	*0.51*
132	July 10, 1904.	2.28	2.28	2.28	2.25	2.18	2.04	1.79	1.62	1.50	1.46	1.37	1.28	*0.96*	*0.77*

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

Paper No. 1071.

MOVABLE BRIDGES.*

By C. C. Schneider, Past-President, Am. Soc. C. E.

With Discussion by Messrs. C. R. Dart, J. R. Worcester, Albert Henry Smith, H. D. Hess, J. E. Greiner, Martin Gay, Ralph Modjeski, Wilbur J. Watson, W. M. Hughes, J. P. Snow, Theodore Cooper, James Christie, J. W. Schaub, George Gibbs, R. Murray, and C. C. Schneider.

The subject of this paper is a review of the mechanical features of movable bridges, and a set of specifications covering their operating machinery or kinetic parts, which the writer respectfully submits for discussion and criticism. The specifications contain the writer's views as to construction and workmanship, also the principles he has followed for his own guide and practice in designing movable bridges. To his knowledge, no specifications of this kind have been published, up to the present time. He hopes that this paper will bring out a full discussion from those who have had experience with movable bridges and other mechanical work covered by his specifications.

A movable bridge is one which may be turned or drawn to one side, lifted up, or let down, so as to permit vessels to pass in the stream which it spans.

Movable bridges may be classified as follows:

1.—Swing bridges, turning about a vertical axis;

2.—Bascule bridges, turning about a horizontal axis, or rolling back on a circular segment;

* Presented at the meeting of April 3d, 1907.

3.—Traversing or retractile bridges, moving horizontally;
4.—Lift bridges, lifting vertically;
5.—Transporter or ferry bridges, consisting of a fixed span with a suspended traveler.

As belonging to the movable bridges, may also be mentioned the pontoon or floating swing bridge. As it is hardly likely that this type of bridge will be much used in America, it will not be considered in this paper.

1.—Swing Bridges.

The swing bridge consists of the superstructure attached to and turning around a pivot, and provided with some means whereby, when closed, the deflection of the girders may be taken out and the ends supported at the proper level. The pivot may be in the center of the span, forming a truss of two equal arms which balance each other and give two openings for navigation; or it may be at one side of the center, forming a truss with unequal arms, in which case the short arm is counterweighted to balance the bridge about its pivot. They may be single or double; in the latter case, they will have to be provided with a locking arrangement at the center where the arms meet, while the shore ends will have to be held down or anchored to the masonry when the bridge is closed, acting as cantilevers. Swing bridges are divided into three classes: Center-bearing, rim-bearing, and a combination of the two.

The Center-Bearing Bridge.—This bridge, while swinging, is carried entirely on the center pivot. The trailing wheels, which roll on a track near the outer edge of the pier, are not assumed to carry any load, but are used to balance the bridge and insure lateral stability while swinging. In short, light, highway spans the portion of the live load which comes on the center when the bridge is closed is sometimes also carried by the pivot. In more important highway bridges, and in all railroad bridges, however built at the present time, the live load at the center is carried on wedges or other independent supports on the pivot pier. In some cases center-bearing bridges of small spans are supported and balanced by the center pivot while swinging—dispensing with balance wheels. They are generally lifted bodily at the center by hydraulic pressure to clear the end bearings.

The Rim-Bearing Bridge.—This bridge, while swinging, is carried by a circular girder or drum on a concentric ring of conical rollers which roll on a corresponding roller path or track; the center pivot receives no load. The rollers also carry the live load at the pivot pier when the bridge is closed.

The less important types of swing bridges are: The shear-pole draw and the jack-knife draw.

The Shear-Pole Draw.—This bridge has only one leaf turning around a pivot at one end; the other end, while swinging, is suspended from the top of a two-legged shear-pole by rods which are attached to a pivot which is vertically in the same line as the pivot below. The shear-pole is stayed by guy rods from the shore end. When the bridge is closed it forms a simple span supported at both ends.

The Jack-Knife or Folding Draw.—This bridge differs from the shear-pole draw in that it consists of two or more separate trusses or girders, each girder swinging horizontally around its own pivot at one end. The other end, while swinging, is suspended from a frame or tower by rods attached to pivots on top of the frame. The rails are fastened to the top of each girder; there are no ties. The girders are connected with hinged links, and fold up close to each other when the bridge is fully opened. When the bridge is closed, one end of each girder rests on the shoe at the abutment, while the other end rests on the pivot. This type of bridge can only be used for railroad traffic.

2.—Bascule Bridges.

This type has been productive of a greater variety of designs than any other kind of movable bridge. Some are designed to rotate about trunnions at one end, the other end being lifted by hoisting ropes from a tower; some are turned by gearing on a segment; some roll back on circular segments, and others have a combined motion of turning and rolling. Nearly all the mechanical motions and devices have been used in designing operating mechanisms and machinery for bascule bridges. The various patented designs offered are so numerous and of such variety as to preclude their mention. Bascule bridges are made in one leaf, or in two leaves which meet in the center. The two-leaf bridges have a locking device at the ends, and are arranged to act as cantilevers when closed, and

sometimes as three-hinged arches. All bascule bridges are counterweighted to reduce the power required for operation. Some very ingenious arrangements of counterweighting have been devised to balance the weight of the span in all stages and positions during opening and closing. The writer hopes that those who have meritorious designs to offer will submit them and point out their advantages, in the discussion of this paper.

3.—Traversing or Retractile Bridges.

Traversing or retractile bridges consist of a simple span across an opening, extended some distance on one side over the abutment. This portion is supported on wheels, and is counterweighted, the overhanging portion acting as a cantilever when the bridge is moving, and resting on the opposite abutment when closed. Traversing bridges are constructed so as to be capable of being rolled horizontally backward in the direction of the center line, or transversely to the center line of the bridge. In the latter case the bridge generally crosses the opening on a skew. In some cases the bridge rolls transversely upon its carriage until it has passed and cleared the track, and then rolls backward to clear the waterway.

4.—Lift Bridges.

A lift bridge consists of a simple span resting on abutments when closed. To the ends of the girders are attached ropes or chains which pass over sheaves on top of a frame at each end of the bridge, by which the vertical motion is accomplished. They are counterbalanced to reduce the power required for lifting.

5.—Transporter or Ferry Bridges.

This type consists of an elevated fixed span supported on towers on each side of the stream. From this fixed span is suspended a platform or car which travels across the opening and thus conveys the traffic from shore to shore. The elevation of the fixed span is such as to clear navigation. The first bridges of this kind were of the suspension type, with a car suspended by wire rope. The only example of this type in America is the ferry bridge across the ship canal at Duluth, Minn., of about 390 ft. clear span. For the fixed span, riveted steel trusses are used, and the car or traveler suspension is of a substantial riveted construction.

Selection of Design.

It is impossible to lay down any hard and fast rule as to which kind of bridge is best adapted for a certain location and under certain conditions; there are so many factors entering into the problem that even the experienced designer is often at a loss to decide on the proper kind of structure.

The writer, in his own practice, has always observed the following fundamental principles in selecting a design for any kind of movable bridge:

1.—When the bridge is closed, and ready to carry traffic, it should be as nearly as possible a fixed span, complying with the requirements of good practice for a permanent structure.

2.—The operating machinery should be designed so that the bridge can be easily handled and operated while moving. The most simple design which gives the least frictional resistance, and also the least first cost and cost of subsequent operation and maintenance, should receive the preference.

3.—The structural and machinery parts of the bridge should be separate and distinct. That is, when the bridge is closed, acting as a fixed span, the machinery parts should receive no strain; and only when the bridge is moving should the machinery parts be in service.

When there are no restricting conditions, a swing bridge is generally the most satisfactory. It is best adapted for long spans; and, for spanning two equal openings, it is the most economical and simple of any. Swing bridges have been built for single- and double-track railroad structures up to 520 ft. span, and for four-track structures up to 390 ft. span, and may be used for spans of much greater length. If there are no other restricting conditions, a swing bridge with equal arms should be selected. If the conditions are such that the pivot pier has to be located at or near the shore, with limited room back of the shore end, a bridge of unequal arms will be best adapted, provided space is available for swinging the bridge.

As stated before, the usual types are center-bearing and rim-bearing swing bridges. The center-bearing type, designed in accord-

ance with good modern practice, offers more advantages than the rim-bearing type, and should always receive the first consideration in determining upon a design. It requires less power to turn, has a smaller number of moving parts, is less expensive to construct and maintain, requires less accurate construction than the rim-bearing bridge, and does not as easily get out of order. The structural and the operating or machinery parts are entirely separate, and when the bridge is closed it forms either two independent fixed spans, or a fixed span, continuous over two openings, resting on firm, substantial supports. There are no ambiguities in the calculations in reference to the distribution of the load, and the distance required from base of rail to masonry is generally less than that required for a rim-bearing bridge with proper distribution of the load over the drum. Any irregular settlement of the masonry does not materially affect its operation.

On the other hand, the rim-bearing bridge requires a circular girder or drum of difficult and expensive construction, a ring of accurately-turned rollers, and circular tracks, which require great care in their construction and delicate adjustment in their erection in order to make the bridge operate satisfactorily. Repairs are troublesome and expensive, and any irregular settlement of the masonry will throw the whole turning apparatus out of order.

The only limit to the center-bearing type is generally the width of the bridge. As the entire weight of the bridge while swinging is carried on the cross-girders to the center pivot, these girders—if the width is excessive—may become so long, and the weight they have to carry so great, as to make their construction impracticable. Center-bearing bridges are adapted for single-track structures of any span, and for double-track they have proven satisfactory up to 323 ft. total span. The writer has also used the center-bearing type for a three-track bridge of 200 ft. total span. For four-track bridges of long span, and for heavy highway bridges carrying wide city streets, the center-bearing type is not as well adapted.

In the center-bearing swing bridge of the modern type of construction, the entire load while swinging is carried on the pivot; the trailing wheels are adjusted so as to have a little clearance. When the bridge is closed, the trusses at the center are borne on wedges or other substantial supports. These supports are not sup-

posed to lift, but should only come in contact; they carry practically all the live load. The center supports and end lifts should be connected so as to work simultaneously, and adjusted so that, when the center supports come to a bearing, the ends are lifted to their proper level. For the center supports the writer uses wedges with a bevel of 1 to 10. The trailing wheels he has used in connection with this type of bridge are from 15 to 20 in. in diameter and from 4 to 6 in. in tread. He generally uses eight wheels, two on each side of the center line of the floor system to balance the bridge transversely, and two pairs at right angles to them to balance the bridge longitudinally, while swinging. It is his practice to proportion the diameter, width of tread, size of axles, and journals of the wheels on each side of the track for an overturning wind pressure of 20 lb. per sq. ft. while swinging, and make the other wheels duplicates of the same.

There are two methods of carrying the weight on the center pivot of a swing bridge: by suspension or by superposition. The advantages of the suspension method are that it brings the point of support nearer the center of gravity of the bridge, that the disks or rollers can be readily taken out, examined, or replaced without stopping the traffic over the bridge, and, also, that it affords an easy means of adjusting the height. Several kinds of pivots have been used for swing bridges: with disks, conical rollers and balls. In the writer's experience, disks have given the best results. Conical rollers have proven unsatisfactory, and have given more or less trouble in many cases where they have been used. As the frictional resistance in rollers is approximately the same as in disks, the pivot with conical rollers has no advantages, and, therefore, is not to be recommended. The best results have been obtained with a phosphor-bronze disk between two hardened-steel disks. As the surfaces of the hardened-steel disks in contact with phosphor-bronze will not wear out, the wearing is confined to the phosphor-bronze disk, which is, therefore, practically the only part outside of the operating machinery which will in time wear out and have to be replaced.

Ball bearings have been used in some cases, and it is claimed that they have proven satisfactory on account of the great reduction of friction. The writer has not been able to obtain any reliable data on this subject.

If the conditions are such as to make a center-bearing swing bridge impracticable, the rim-bearing type has to be used.

The most important requirement for a satisfactory rim-bearing swing bridge is to have the load equally distributed over all the rollers. If this is not accomplished, some rollers will receive more load than others, the drum with the upper track will deflect between the points of support, and the bridge will turn hard, and, consequently, there will be a great deal of unnecessary wear and tear on the rollers and track. In order to obtain an equal distribution, it is necessary to have practically no deflection in the drum between the points of support. To effect this, the points of support should be close together and the depth of the drum relatively great. In the writer's judgment, the depth of the drum should be not much smaller than one-half, in no case less than four-tenths, of the distance between the centers of support. Many rim-bearing swing bridges for single-track have been built with only four points of support on the drum. This is inadequate for good distribution; they should have at least eight points, and double-track bridges preferably more. The distribution should be precise, and should be arranged so that it will not be affected by the deflection of the distributing girders.

Rim-bearing swing bridges in some cases give trouble on account of the center pivot working loose, as a result of inaccurate workmanship or adjustment, or by bad masonry. For this reason, it is an advantage to have a certain portion of the weight on the pivot while the bridge is swinging, as it assists in resisting any tendency toward lateral movement on the part of the ring of live rollers. If this is done, the arrangement of distribution should be such that the portion of the load which is to be carried by the pivot can be determined accurately. The usual practice in such cases is to carry about one-fifth or less of the entire dead load on the center pivot while the bridge is swinging.

In case it is impracticable to carry a portion of the load on the pivot, the lower circular track should be connected to the pivot with radial struts so that the track and pivot form one piece which can be fitted together and centered accurately in the shop. In order to prevent the center pivot from working loose, some engineers have embedded its base in concrete. This method may keep the pivot in

position, but it does not prevent the pushing, pulling and wearing around the collar of the pivot.

The live-ring arrangement for separating and holding the rollers —which was formerly popular—consisting of adjustable radial rods, one end of each rod carrying a roller, the other end being connected to a ring revolving around the center pivot, is, to say the least, an unmechanical contrivance. Good practice requires a more substantial construction. The rollers should run between two concentric circular girders firmly connected to each other; this ring should be connected to the revolving part of the pivot by rigid struts.

End Lift.—All swing bridges require an arrangement to lift the ends when closed, so as to make the span continuous over three or four supports, or to make two separate simple spans. Various mechanical contrivances are used for this purpose, such as rollers, screws, cams, eccentrics, toggle joints, wedges, hydraulic rams, etc. In the writer's opinion, the lifting apparatus should fulfill the following requirements:

1.—The end lift should have power enough to lift the ends to the desired level with nearly uniform resistance.

2.—After the ends are lifted to their final position, they should form solid, substantial supports, similar to the end shoes of a fixed span.

That kind of end lift which supports the ends of the bridge, when closed, on rollers, toggle joints, or links, is not to be recommended. The writer has found wedges with a bevel of 1 to 5 or 1 to 6 the most satisfactory. The mechanism for moving the wedges can be arranged so as to offer nearly uniform resistance in all stages of the lifting, and also to lock the wedges to prevent them from sliding backward. They form as substantial supports for the ends under traffic as can be desired. Toggle joints and hydraulic rams have also proven satisfactory, if supplemented by wedges or other substantial supports, after the lifting is accomplished.

In center-bearing bridges of moderate span, with unequal arms, especially when the short arm is over the abutment, it is of advantage to have the lifting apparatus only under one end. This end should have an excess of weight, so that, when the end supports are removed, the bridge will tilt and throw some weight

on the balance wheels. The opposite ends of the girders simply rest on bed-plates. This arrangement dispenses with one-half of the lifting machinery.

Such tilting arrangements may also be used in case the lifting is done in the center by a hydraulic ram under the pivot.

Some swing bridges are designed with sufficient lift at the ends to make the trusses discontinuous, so that they will form two separate simple spans when the bridge is closed. The operation of this requires approximately ten times as much work as if the ends were lifted only enough to prevent the lifting of the ends from their supports under traffic and their consequent hammering. This arrangement, therefore, is a disadvantage in a bridge which has to be opened and closed frequently and quickly; in fact, it is only a waste of power, but may be recommended in cases where the bridge is used almost entirely as a fixed span, and where the time required for the opening is of no importance. Lifting the bridge from the center is not as economical as lifting at the ends, on account of the greater power required. A few bridges have been built with a lifting apparatus applied at the top chord of the center panel. This arrangement has the additional disadvantage of using a part of the structure which carries the strain while the bridge is swinging also as a part of the lifting mechanism, which is not good practice. If the machinery parts get out of order, the structure should not be affected thereby. The writer's experience with a structure of this kind prompts him to advise against its use.

The Shear-Pole Draw, and the Jack-Knife Draw.—These also come under the head of swing bridges, but can only be used for very small spans. They are not to be recommended, except as temporary makeshifts on new roads, or in places where there is little navigation.

Bascule Bridges.—If the conditions are not favorable for a swing span, a bascule bridge is the next choice. The conditions which preclude the use of a swing bridge are:

If there is no space available to swing the bridge horizontally about its vertical axis;

If the number of tracks is such as to increase the width of the bridge and the size of the pivot pier to such an extent as to make it undesirable or impracticable; or,

If there is a probability that other tracks will have to be added in the future.

The turning space required for swing bridges is a disadvantage, and increases with the size of the bridge. The bascule bridge, on the other hand, has the advantage that it can be used in places where there is no space available alongside of the bridge. The bascule bridge can be enlarged or widened by putting up additional spans alongside of it, without interfering with the operations of the existing span or with navigation. The length of span, however, is somewhat limited, as the question of wind pressure becomes very important in long spans, and constitutes one of its greatest disadvantages.

The limiting length of clear span is probably 150 ft. for single-leaf, and 300 ft. for double-leaf bridges.

The design which should be selected for a bascule bridge depends upon many conditions, such as location, distance from the floor to water level, under-clearance required, length of span, frequency of opening, speed required, kind of power available for operating, etc. In determining the type and details of the design which will be most suitable for the existing conditions, the principles mentioned before, which govern all kinds of movable bridges, should be considered.

Traversing or Retractile Bridges.—There are some objections to the traversing bridge, which make it undesirable, especially for railroad traffic. It requires more power to move it than almost any other kind of movable bridge, and is slow of motion. It has been used in only a few cases for railroad bridges, but has proved satisfactory for small highway bridges. It has been used successfully in some locations where no other type would answer, as it fits certain conditions better than any other design.

Vertical-Lift Bridges.—The vertical-lift bridge has the following advantages over the bascule bridge:

It can be made of any length feasible for a simple span, while the span of a bascule bridge is somewhat limited. The bridge, when closed and carrying the traffic, forms a simple fixed span, fulfilling all the requirements of a permanent structure. Its disadvantages are heavy first cost and maintenance, and expensive

operation. The vertical-lift bridge is most suitable for spans which require only a small lift, such as bridges over canals.

Transporter or Ferry Bridges.—Such bridges take the place of ferries. They have the advantage over the ferry-boat that their operations are not obstructed by ice; therefore they can be used in cold climates all the year round, where the operation of ferries would have to be discontinued during the winter months. Their capacity, however, is very limited, and they are not suitable for railroad uses.

Power Required to Operate Movable Bridges.

The determination of the power required to operate movable bridges is not a matter of elaborate scientific calculation, but of practical experience and judgment. One should be guided in a general way, of course, by the experiments which have been made to ascertain the frictional resistance in different kinds of movable bridges; but, as the conditions do not remain the same, the frictional resistance varying in different bridges even of the same type and construction, and as the machinery sometimes rusts up, a wide margin for the required power should be allowed. It is always an advantage to have an excess of power, as it adds only a very small percentage to the total cost of the structure, does not increase the operating expenses, and increases the efficiency and reliability of the operating machinery.

The resistances to be overcome in turning a swing bridge are:

1.—Resistance due to friction;
2.—Resistance due to the inertia of the bridge;
3.—Resistance due to the action of the wind.

1.—Resistance Due to Friction.—For rim-bearing swing bridges, the late C. Shaler Smith, M. Am. Soc. C. E., found the total frictional resistance in the live ring to vary from 0.004 to 0.008 of the load on the rollers. Messrs. Boller and Schumacher found this coefficient to be 0.0035 for the Thames River Bridge; and Theodore Cooper, M. Am. Soc. C. E., found this to be 0.0038 for the Second Avenue Bridge. For center-bearing bridges, Mr. C. Shaler Smith found the frictional resistance at the circumference of the pivot to be 0.09 of the weight turned. The writer found the coefficient

of frictional resistance at the circumference of the pivot, on hardened-steel and phosphor-bronze disks, with the usual working pressure of about 3 000 lb. per sq. in., to be 0.067 at the start, and 0.045 to keep the bridge moving at a uniform speed. For the total frictional resistance, including that of the shafts and gearing required for hand-operation, the highest coefficients observed were 0.115 for starting and 0.08 for keeping the bridge in motion. These experiments were made on new bridges, before the gearing, etc., had an opportunity to wear smooth. For bridges which have been in operation for some time, these coefficients would be found to be considerably smaller.

2.—Resistance Due to the Inertia of the Bridge.—The power required to overcome the inertia of the swinging mass and develop the desired velocity depends upon the time allowed for opening or closing. It is the usual practice to assume that the acceleration is developed in half the time it takes to open the bridge. With this assumption, the following approximate formulas will give results near enough for practical purposes:

$$P = \frac{W \rho}{20.5\ t^2};$$

$$H\text{-}P = \frac{P\ v}{550} = \frac{W\ \rho^2}{7\ 170\ t^3};$$

P = Force applied to the center of gyration necessary to produce the required acceleration;

H-P = Number of horse-powers;

W = Total moving weight, in pounds;

l = Length of bridge, in feet;

b = Width of bridge, in feet;

ρ = Radius of gyration $= \sqrt{\frac{b^2 + l^2}{12}}$, approximately $0.29\ l$;

t = Time, in seconds, in which maximum velocity must be obtained, or one-half the number of seconds required to open the bridge;

v = Maximum linear velocity at the center of gyration, in feet per second $= \frac{\pi \rho}{2\ t}$.

3.—Resistance Due to the Action of the Wind.—Some engineers assume an unbalanced wind pressure of 4 or 5 lb. per sq. ft. in de-

termining the power required to overcome the resistance caused by the wind. This unbalanced wind pressure is considered to act entirely on one arm of the bridge at right angles, and follow the bridge in its various positions while swinging. This is an arbitrary assumption which has never been verified. It is known, however, from actual observation, that with a strong wind it may require nearly twice the power to turn the bridge than without wind. It is suggested, therefore, that the resistance due to the action of the wind be included in the coefficient of friction.

This, of course, applies only to swing bridges with two equal arms. If the bridge has arms of unequal length, the resistance due to wind pressure should be calculated and included in the total resistance.

For rim-bearing bridges, the writer's practice has been to assume as the total resistance to be overcome in turning, including frictional resistance of the gearing and wind pressure, about twice the value given by Mr. C. Shaler Smith for frictional resistance, which value was also recommended later by Messrs. Boller and Schumacher, *viz.*, 0.015 of the load on the rollers acting in the center line of the track; and, for center-bearing bridges, 0.15 of the load on the pivot, when swinging, acting on the circumference of the pivot. In estimating the capacity of the motor, he assumes a linear velocity, in the center line of the track, of twice that required for turning the bridge in the specified time; or, as expressed by formulas:

$P = 0.015$, for rim-bearing;

$P = 0.15\ W \frac{r}{R}$, for center-bearing;

$$H\text{-}P = \frac{P\,v}{550};$$

$$v = \frac{R\,\pi}{2\,t};$$

P = Force required to turn the bridge, acting in center line of track;

W = Total moving weight, in pounds;

$H\text{-}P$ = Number of horse-powers;

r = Radius of pivot, in feet;

R = Radius of center line of track, in feet;

v = Maximum linear velocity in center line of track, in feet per second;

t = Time, in seconds, in which maximum velocity must be obtained, or one-half the number of seconds required to open the bridge.

The foregoing formulas for frictional resistance generally give results in excess of the power actually needed under ordinary conditions, and practically cover all contingencies.

It is supposed, of course, that the bridges are of rational construction and average workmanship.

If the bridge is to be operated by hand-power, the gearing has to be arranged, either for the number of men available, or for the time required to turn and operate the bridge. Mr. C. Shaler Smith recommended the assumption of the power of one man on a capstan bar as 40 lb. at a speed of 250 ft. per min. The writer found that this velocity is somewhat high, and does not fit all sizes of men, but that the work which can be performed conveniently by any ordinary man for considerable time is about 40 lb. with a speed of 160 ft. per min. In calculating the strength of the gearing, etc., the power of one man should be taken at 125 lb., as this is about the force a strong man, with a foothold, can exert for a short time.

For bascule and lift bridges the writer has not been able to obtain any data in reference to frictional resistance and power required for operating, but experiments may have been made, and he hopes that their results will be brought out in the discussion of this paper.

In bascule and lift bridges, the wind pressure is an important factor in increasing the frictional resistance of the machinery, but the pressure may be taken as much smaller than that usually specified for the structural parts of the bridge, as navigation would be impossible with such a wind, so that the bridge would not require to be opened. The writer believes that 20 lb. per sq. ft., corresponding to a velocity of about 60 miles per hour, is sufficient for all cases.

For swing bridges, it has been the usual practice to use a foot-brake to control or stop the motion of the bridge. This, no doubt,

is a good precaution, where it can be conveniently arranged, as in cases where the motor is located in the operator's house. However, when the bridge is operated by electricity, the motors on the shafts of the operating machinery are not generally within reach of the operator, and the introduction of a foot-brake involves considerable complication. For this reason, the writer has omitted brakes in all swing bridges operated by electricity, and this has proved satisfactory in all cases.

Motors.

In the past, up to about fifteen years ago, steam and hydraulic power were the only motive powers used for operating movable bridges. In later years, electric and internal combustion motors have in many cases taken the place of the steam engine.

Electric motors are the most convenient, where electric power can be obtained satisfactorily, more particularly if the company owning the bridge also controls the power, such as is the case with electric railway bridges, or steam railroad bridges situated near one of the company's power-houses, or city bridges. Electric power is economical, and the motors occupy little space and can be put in the most convenient places, so as to reduce the transmission by long shafts.

The requirements for electric motors and electrical equipment given in the subjoined specifications are based on those adopted by the Sanitary District of Chicago for their bridge crossing the Chicago River at Dearborn Street, which appeared to be the most complete of any which have come to the writer's notice. They have been modified so as to make them general, and applicable to any kind of movable bridge.

Where electric power cannot be obtained, a steam engine or internal combustion engine has to be used, excepting in special cases, where water-power may be used to advantage. For large spans, which have to be opened frequently and quickly, a steam engine is preferable. However, it has its disadvantages. It occupies more room than almost any other motor, as it requires storage capacity for coal and water, machinery for hoisting coal and pumping water, and the ashes have to be disposed of. The

fire under the boiler has to be kept up when the bridge is not in operation, and the operating expenses are high.

For smaller spans and intermittent service, the gasoline engine is better adapted. It is more economical, and requires less skill to operate than the steam engine. The internal combustion engine has been brought to a high degree of development, particularly in the smaller sizes, and many of the defects of the earlier engines are now almost eliminated. Gasoline engines, up to 50 h. p., have been used successfully for operating movable bridges.

So many different types of gasoline and other internal combustion motors have come into the market in the last few years, such as vertical and horizontal engines, engines with one, two and three cylinders, two-cycle and four-cycle motors, etc., that engineers who are not experts in internal combustion engines are generally puzzled when called upon to select the most suitable engine for the work to be performed in operating a movable bridge. The writer hopes that this subject will be discussed by engineers familiar with the latest developments in these engines.

Hydraulic power can be used to advantage in exceptional cases only, where the bridge is operated from the shore, and where water with sufficient pressure can be obtained for working a hydraulic motor.

Hydraulic power, however, is generally used with hydraulic rams in connection with accumulators. They have the advantage that a great amount of power can be accumulated with a small motor during the time the bridge is not being operated. Arrangements of this kind have been used successfully on a number of bridges.

Unit Strains.

The permissible unit strains for machinery parts in tension, compression, bending and torsion, given in the subjoined specifications, are those recommended by Professors Von Bach and Unwin, slightly modified to conform to the writer's views. The safe bearing values given for rotating and sliding surfaces agree with those now generally considered good practice in machine design, and have been used extensively and verified by the writer.

The safe bearing pressures on rollers are based on experiments made by James Christie, M. Am. Soc. C. E.

The values of the permissible pressure on ball bearings of hardened steel are those recommended by R. Stribeck as the result of his experiments, made at the Prussian Government Laboratory, on balls and races manufactured abroad. These values, therefore, are not applicable to ball bearings manufactured in the United States, about which the writer was not able to obtain any information.

Workmanship and Material.

The quality of workmanship called for in the specifications is the same as the writer has required for machinery parts of movable bridges, when he has had control of the shops where the work was being done.

The specifications for special metals, generally not covered by specifications for the structural work of bridges, are practically those adopted by the American Society for Testing Materials. For tool steel, phosphor-bronze and babbitt metal, for which that society has no specifications, the writer has given what he has found satisfactory in his own practice.

SPECIFICATIONS FOR MOVABLE BRIDGES.

Types of Bridges.

Movable bridges, of the kind used in modern practice for carrying railroad traffic, are generally of the following types:

1.—Swing Bridges;
2.—Bascule Bridges;
3.—Vertical Lift Bridges.

The type of bridge best adapted for a particular location depends upon local conditions, and should be left to the judgment of the engineer.

Loads, Unit Strains, etc.

The specifications for loads, unit strains, proportion of parts, material and workmanship adopted for fixed spans will also apply to the structural work of movable bridges under the various conditions of loading.

Wind Pressure.

The minimum wind pressure to be assumed in proportioning the machinery or moving parts of movable bridges, such as trailing wheels on center-bearing swing bridges, operating machinery of swing bridges with unequal arms, and bascule bridges, etc., shall be 20 lb. per sq. ft. on the exposed surfaces of all trusses and the floor system as seen in elevation.

Swing Bridges.

Types of Girders.

The following types of girders and trusses are recommended:

Plate girders, up to 150 ft. span;
Riveted trusses, from 150 to 250 ft. span;
Pin-connected trusses may be used for spans greater than 250 ft.

Conditions of Loading.

Trusses of swing bridges which are continuous over three or four supports shall be calculated for the following conditions:

1.—Bridge open, unloaded;
2.—Bridge closed, no reaction at end from dead load;
3.—Bridge closed, ends lifted to make trusses continuous.

Turn-Tables.

The turn-table may be center-bearing, rim-bearing, or a combination of the two. If a combination table is used, the supporting girders shall be arranged so that some definite portion of the load will be carried to the center. The center-bearing type of turn-table should preferably be used, whenever practicable, and is recommended for single-track bridges up to 500 ft., and for double-track bridges up to 400 ft. span.

Center-Bearing.

Center-bearing turn-tables should be designed so that the entire dead load of the bridge is carried on a center pivot when the bridge is swinging; but, when the bridge is closed, the trusses should rest

at the center on wedges or other substantial supports. These supports must be strong enough to resist the reaction resulting from the live load and impact of the train. Trailing wheels running on a circular track should be provided for the purpose of balancing the bridge and carrying the wind pressure to the track while swinging. The center pivot, as well as the trailing wheels, should be adjustable as to height. The center pivot should be designed so that the disks or rollers can be taken out, examined, or replaced, when the bridge is not in operation, without interfering with the railroad traffic.

Rim-Bearing.

Rim-bearing turn-tables, which carry the bridge on a drum rotating on rollers between two tracks, shall have the center pivot connected to the drum by rigid struts. The object of this pivot is to center the turn-table and hold it in place while the bridge is swinging. The drum shall be designed so that the load is distributed properly over all the rollers. The rollers shall be of such size and number as to carry the entire dead load when the bridge is swinging, and the reaction of the dead and live load and impact when closed. The lower track shall be strong enough to distribute the loads on the rollers over the masonry. The drum shall be designed in accordance with the specifications for plate girders. It shall be provided with effective stiffeners or fillers on both sides of the web at all points of concentrated loading; these stiffeners shall have a close bearing against the upper and lower flanges. The bottom of the lower flange should be planed. The lower track shall be anchored to the masonry with bolts, not less than $1\frac{1}{2}$ in. in diameter and 15 in. long, set in Portland cement grouting.

Rack and Track.

The rack and track segments shall be made in short sections, preferably not more than 5 ft. long. If a cast lower track is used, in case the track is light, as in center-bearing bridges, the rack and track shall preferably be cast in one piece. The maximum pitch of the teeth in the rack shall not exceed 6 in. For small bridges, steel rails may be used for the track.

Pinions.

If the pressure requires a greater pitch than 6 in., two or more pinions shall be used, and they shall be arranged so that the pressure will be distributed equally over all pinions. If two pinions are used, they shall be placed opposite one another.

Brackets

The brackets to support the pinions gearing into the rack must be made in one piece, so that both bearings can be bored true in line and will not get out of adjustment. They shall be provided with caps so that the pinion shaft can be taken out without removing the bracket.

End-Lifting Apparatus.

All swing bridges must be provided with an effective lifting and locking apparatus at the ends. The lifting apparatus must be strong enough to exert an uplifting force equal to the maximum

negative end reaction of the live load plus 50%, and must have a bearing capacity for the positive maximum end reaction, including impact.

Rail-lifts shall be provided, in connection with the end-lifting apparatus, as well as automatic self-acting latches. The latches must be arranged so that they will lock the bridge automatically when in the proper position for closing, but only if the speed is so slow that the shock will do no damage. The end supports should be arranged so that they can easily be adjusted to the proper amount of lift.

Signals. Bridges carrying railroads shall have at each shore end an automatic signal, and such signals shall be arranged so that the bridge cannot be opened without setting each signal at the danger position, and so that the signals cannot be set to indicate a clear track until the bridge is closed and the rails are in their proper position.

Bascule and Lift Bridges.

The general principles governing the design of swing bridges are also applicable to lift bridges of any kind, as well as to other movable bridges.

Operating Machinery.

Details of Design.

General. The operating machinery of movable bridges shall be designed and constructed in a substantial manner: all complicated and flimsy contrivances shall be avoided. All parts shall be designed so that they may be easily erected, adjusted and taken apart, and must be accessible for inspection, cleaning and repairs. The fastenings shall be designed so that after all machinery parts are properly set, lined up and adjusted, they will be permanently fixed.

Materials. *Rolled or Forged Steel.*—Rolled or forged steel shall be used for bolts, nuts, keys, cotters, pins, axles, shafts, screws, worms and piston rods.

Forged or Cast Steel.—Forged or cast steel may be used for levers, cranks and connecting rods.

Cast Steel.—Cast steel shall be used for pivot stands, couplings, wedge-bearings, rollers, trailing wheels, end shoes, and for racks, tracks and pinions, and toothed wheels of bridges operated by mechanical power.

Cast Iron.—Cast iron may be used for journal boxes, pulleys, drums, eccentrics, and toothed wheels for bridges operated by hand-power only; also for cylinders, pistons, fly-wheels, brake-wheels, and other parts of motors which are usually made of cast iron.

Hardened Steel.—Hardened steel shall be used for pivot disks,

friction-rollers, ball-bearings, and in other similar cases, for the purpose of reducing the bearing surface, abrasion and friction.

Phosphor-Bronze, Brass and Babbitt Metal.—These or other alloys shall be used for the bushing or lining of journal bearings and other rotating or sliding surfaces to prevent seizing, which is likely to occur when steel moves on steel.

Castings. All castings which are to be attached to rough, unfinished surfaces shall be provided with chipping strips. The outer unfinished edges of all ribs, bases, etc., shall be rounded off, and inside corners shall have fillets.

Bolts and Nuts. All bolts and nuts, up to 1½ in. in diameter, shall have U. S. Standard V-threads. All nuts and exposed bolt heads shall be of hexagonal shape, and each nut shall be provided with a washer. If the nut bears on an inclined surface, the washer must be beveled. Bolt heads which are countersunk in a casting shall be square.

Lock-Nuts. Nuts which are subject to vibration and frequent changes of load shall have locking arrangements to prevent the gradual unscrewing of the same. If double nuts are used for that purpose, each nut shall be of the standard thickness.

Screws Transmitting Motion. All screws which transmit motion must have square threads.

Tap Bolts. Tap-bolts shall be avoided as much as possible.

Set-Screws. Set-screws shall not be used for fastening wheels, pulleys, pinions, cranks, or any other parts which transmit torsion to shafts or axles. They shall only be used in connection with keys, and for fastening collars to shafts in cases where the collar is not subject to any strain in the direction of the axis of the shaft.

Collars. Collars shall be used wherever necessary to hold the shafting from moving horizontally. Each collar shall have at least two set-screws, set at an angle of 120° to one another.

Keys. All pieces which transmit torsion to shafts or axles shall be fastened with gib-head keys having a taper of ⅛ in. to 1 ft., and shall be seated in grooves, in both hub and shaft. If two keys are used, they shall not be placed opposite, but at an angle of 120° to one another. The minimum width of the key shall be one-fourth of the diameter of the shaft if one key is used, and one-sixth, if two keys are used, and the minimum thickness shall be one-half the width.

Journals. Journals shall be proportioned to resist, not only the various strains to which they are subjected, without exceeding the permissible fiber and bearing strains, but also to prevent a tendency to heat and seize.

Bearings. Steel bearings carrying steel shafts or journals should be babbitted, or have a lining of some other material, preferably bronze. Bearings of steel on steel shall not be used on any rotating or sliding surface, unless provided with metaline plugs, or unless one or preferably each of the surfaces is hardened. The bearings of shafts should be placed as near to the points of loading as possible.

Pivots. Pivots for the centers of turn-tables may revolve on disks, friction-rollers, or balls. Disk-bearings shall preferably consist of three disks, one of phosphor-bronze between two disks of hardened steel, so that the steel will slide on the bronze. The phosphor-bronze shall be of the special kind specified on page 295. Friction rollers or balls shall be made of hardened steel, and they shall run on bearings of the same material. Rollers shall be separated by a spider to prevent them from coming in contact with each other.

Foot-Steps. The foot-steps of vertical shafts may be of axle or tool steel, and shall run on bronze disks.

Axles. When wheels are used in connection with axles, such as trailing wheels for turn-tables, the wheel must be fastened to the axle and allowed to turn in its journal bearings. Wheels shall not be allowed to rotate loose on any axle.

Lubrication. Provision must be made for the proper and effective lubrication of all journals, pivots, or any other moving parts with sliding or rotating surfaces. Closed oil or compression grease cups shall be provided for all journal bearings; where not otherwise accessible, they shall be connected with oil pipes. Oil grooves must be provided wherever necessary for the proper distribution of the lubricant. All oil holes must be easy of access.

Dust Covers. Dust covers shall be provided wherever necessary to protect the sliding and rotating surfaces and prevent dust from mixing with the lubricant.

Shafts. Line shafts, on account of the flexibility of the structure to which they are attached, shall not be continuous, but shall be connected with flexible couplings. Each length of shafting should rest preferably in two regular bearings only, with the couplings on the outside and close to the bearings in opposite directions. In case the bearings or journal boxes are attached to the floor-beams, it may become necessary, in bridges with long panels, to provide intermediate supports in order to reduce the deflection of the shaft. These intermediate supports shall consist of loose-fitting open bearings, the functions of which shall be to prevent the shaft from sagging. The unsupported length of shafting shall not exceed:

$l = 80 \sqrt[3]{d^2}$ for shafts supporting their own weight only;

$l = 50 \sqrt[3]{d^2}$ for shafts carrying pulleys, gearing, etc.;

where l = length of shaft between bearings, in inches; and d = diameter of shaft, in inches.

Line shafts connecting the machinery at the center to that at the ends should run at fairly high speed. The requisite speed reduction should be in the machinery near the end.

Couplings. Couplings to connect shafting shall be claw couplings, and shall be placed as near the bearing as possible. Each half of a coupling shall be fastened rigidly to the end of the shaft, but the

coupling must not be bolted together, in order to allow slight angular motion without twisting and binding the shaft in its bearings. They shall be strong enough to develop the full strength of the shafts to which they are attached.

Toothed Gearing.

Gear wheels should be designed on the assumption that one tooth transmits the whole pressure. In uncut gearing, the pressure should be assumed as coming on a corner of the tooth; but, in machine-cut gearing, it should be assumed as distributed over the whole width of the tooth. All cast pinions shall be shrouded, and no pinion shall have less than twelve teeth. All toothed gearing in operating machinery shall preferably have involute teeth. The minimum width of the teeth should be one and one-half times the pitch; for uncut teeth, generally, the width should be two times the pitch. For cut gearing, the width may be greater, but not more than three times the pitch, excepting for wheels moving at a very high velocity, and those in motors, where the wearing and abrasion have to be considered. In estimating the strength of teeth in bevel wheels, the pitch at the inner circumference shall be taken. Bevel wheels shall preferably be avoided.

Screw Gearing.

Screw gearing should be arranged to have proper lubrication between the sliding surfaces of worms and wheels. To accomplish this, the worm should be placed below the wheel and run in an oil bath. The worm shall preferably be of rolled or forged steel; the wheel may be of bronze or cast iron. Worm-wheels shall have not less than twenty-eight teeth.

Wire-Rope Gearing.

If wire-rope gearing is used to transmit power, the diameter of the drum or pulley should be made as large as possible, and not less than 150 diameters of the rope. The bottom of the groove of the pulley shall be lined with leather or hard wood.

Hydraulic Machinery.

In hydraulic rams, externally-packed plungers shall be used. Pistons packed against the bore of cylinders shall be avoided wherever possible. Packing shall be made in the ordinary stuffing-box, with any approved fibrous packing. The depth of the packing shall be at least six times its thickness. The gland shall always be screwed down tight without compressing the packing, which shall be set out by initial pressure alone. When economy of space requires it, leather rings of rectangular section may be used for packing. These shall be soaked in melted paraffine and bored to fit the plunger, or turned externally to fit the cylinder, but shall be free in all other directions. Pipes for conveying fluid shall be disposed so as to have drainage outlets whereby the whole system can be readily drained.

Workmanship.

General.

All workmanship and finish shall be equal to that of the best practice in modern machine-shops. As the parts of the operating

machinery of movable bridges are generally exposed to the weather, the finish shall be confined to the bearing, rotating, and sliding surfaces, and wherever it is required to produce accurate fits and precise dimensions.

Castings. All castings shall be properly cleaned, and all fins, seams, and other irregularities removed, so that they will have clean, smooth surfaces.

Drainage Holes. Drainage holes, not less than $\frac{3}{4}$ in. in diameter, shall be drilled in all places where water is likely to collect.

Bolts. Unfinished bolts may have a play of $\frac{1}{16}$ to $\frac{1}{8}$ in. in the bolt holes. All turned bolts must have the diameter of the shank at least $\frac{1}{16}$ in. larger than the diameter of the threaded portion, and must have a driving fit in the bolt hole.

Track. All track segments shall be planed on both sides and at the joints. The surfaces on which the rollers bear shall be planed to the true bevel, and have the center line plainly scribed thereon.

Rack. Toothed segments forming the rack shall be accurately fitted; particular care shall be taken to have the pitch of the teeth accurate at the joints. The periphery and the upper face of the teeth shall be planed, and the pitch line scribed thereon. If the rack is separate from the track, the rack segments shall be fitted to those of the track, so as to have the center line of the track exactly concentric with the pitch line of the rack.

Rollers. All rollers shall be turned at the circumference and at the faces of the rim, with the corners chamfered and the center line of the roller scribed on the circumference. The hubs shall be accurately bored and faced at each end.

Centers and Pivot Stands. Pivot stands and center castings of swing bridges shall be properly finished and fitted. Particular care must be taken to have the base faced truly at right angles to the axis, and turned on the circumference concentric with the axis.

Pivots. Steel disks, friction rollers or balls used in pivots, as well as their bearings, must be of tool steel, accurately turned and finished to gauge, and oil-tempered. After hardening, they shall be accurately ground to their final finish. Steel and phosphor-bronze disks shall have their sliding surfaces finished to a high polish.

Journals and Bearings. All journals shall be turned with a fillet at each end, and shall have a good workmanlike fit in their bearings.

Hubs. All hubs of wheels, pulleys, couplings, etc., shall be bored to fit close on the shaft or axle. If the hub performs the function of a collar, the end next to the bearing must be faced. Holes in hubs of toothed gear wheels must be bored concentric with the pitch circle.

Toothed Gear Wheels. The circumference of all gear wheels must be turned. All wheels moving with a velocity of more than 3 ft. per sec. at the

pitch line, as well as all bevel wheels, and those in the gearing of any motor, shall have machine-cut teeth.

Screw Gearing.

Threads on worms must be cut, and the teeth of worm wheels must fit the worm accurately.

General.

All machinery which is of the regular standard manufactured type, such as steam, gasoline, electric or hydraulic motors, pumps, air compressors, etc., must be guaranteed by the manufacturer as to efficiency, and shall be subject to the approval of the engineer. All motors shall be tested to prove that they fulfill the specified requirements and develop the desired speed and power. The rating of a motor shall be the horse-power determined by the brake test.

Patents.

The contractor shall indemnify and save harmless the purchaser against all loss or damage, claims and demands, costs and charges, that may arise or accrue by reason of the adoption or use by the contractor of any patented article, device or improvement furnished by him.

Unit Strains.

Permissible Strains.

The unit strains used in designing machinery parts shall not exceed those given in Table 1.

TABLE 1.—Permissible Unit Strains, in Pounds per Square Inch.

Kind of strain, and loading.		Structural steel.	Axle steel.	Steel castings.	Cast iron.	Rolled copper.	Brass.
Tension	(A)	16 000	18 000	12 000	3 000	6 000	3 000
	(B)	10 600	12 000	8 000	2 000	4 000	2 000
	(C)	5 300	6 000	4 000	1 000	2 000	1 000
Compression	(A)	16 000	18 000	16 000	12 000	6 000	3 000
	(B)	10 000	12 000	10 000	8 000	4 000	2 000
Bending	(A)	16 000	18 000	15 000	6 000	.....	.....
	(B)	10 600	12 000	10 000	4 000	.....	.....
	(C)	5 300	6 000	5 000	3 000	.....	.....
Shear	(A)	12 000	14 400	9 000	3 000	3 600	.....
	(B)	8 000	9 600	6 000	2 000	2 400	.....
	(C)	4 000	4 800	3 000	1 000	1 200	.....
Torsion	(A)	10 000	12 000	7 500	3 000	.....	.....
	(B)	6 600	8 000	5 000	2 000	.....	.....
	(C)	3 300	4 000	2 500	1 000	.....	.....

A. For a static load;
B. For a varying load producing strains of tension or compression only;
C. For a varying load producing equal maximum strains in opposite directions, accompanied by shocks and vibrations.

In selecting the working unit strains from Table 1 for designing parts of the operating machinery of a movable bridge, it should be borne in mind that these parts are generally strained in both directions, that the motions are irregular, are frequently accompanied by shocks and vibrations, and that the loads are applied more or less suddenly.

For most machinery parts, such as shafts, axles, gearing, levers, cranks, etc., the figures given for C should be used if the bridge is operated by mechanical power. If hand-power only is used, the values for C, or intermediate ones, may be used, according to the judgment of the designer.

The permissible unit strains for bending (given opposite B)—10 000 for cast steel and 4 000 for cast iron—shall be used for determining the strength of toothed gearing. The teeth in cut gearing shall conform to the following proportions:

$$P = s\,p\,f\,y;$$

where P = pressure on tooth, in pounds;
s = permissible unit strain;
p = pitch, in inches;
f = face of tooth, in inches;
y = a factor established by experience.

For wheels moving at slow speed (up to about 100 ft. per min.), where strength only is to be considered, $y = 0.05$.

The strength of uncut teeth shall be computed for a face of one and one-half times the pitch, or $P = \frac{3\,s\,p^2\,y}{2}$; which conforms approximately with the assumption that the pressure is carried on one corner of the tooth.

For higher velocities, which tend to increase the shock and wear, this value shall not exceed:

$$y = 0.05\,\frac{10}{\sqrt{v}} = \frac{0.5}{\sqrt{v}};$$

where v = velocity in pitch circle, in feet per minute.

For bronze wheels, the same values shall be used as for cast iron.

Fixed Bearings.

For the permissible unit pressures for fixed bearings of different metals, those in Table 1, for compression, shall be used.

Pressure, in Pounds per Linear Inch, on Rollers at Rest.

Cast iron	$400d$
Rolled and cast steel	$800d$

Where d = diameter of roller, in inches.

Permissible Unit Strains for Bearing on Rotating and Sliding Surfaces.

Maximum Bearing Values for Rotating and Sliding Surfaces, in Pounds per Square Inch.

Bearings on Which the Speed is Slow and Intermittent.

Moving Bearings.

Pivots for swing bridges, hardened tool steel on special phosphor-bronze	3 500
Trunnion bearings for bascule bridges, axle steel on phosphor-bronze	2 000
Wedges, cast iron on bronze	600
Wedges, cast iron on cast iron or structural steel	500
Screws which transmit motion on projected area of thread	200

For Ordinary Cases, Parts Moving at Moderate Speeds.

Hardened steel on hardened steel	2 000
Hardened steel on bronze	1 500
Tool steel (not hardened) on bronze	900
Structural steel on bronze	600
Cast iron on structural steel	400
Cast iron on cast iron	400
On cross-head slides, speed not exceeding 600 ft. per min.	50

In order to prevent heating and seizing at higher speeds, the pressure on pivots or foot-step bearings for vertical shafts and journals shall not exceed:

$$\text{On pivots} \ldots\ldots p = \frac{160\,000}{n\,d}$$

$$\text{On journals} \ldots\ldots p = \frac{300\,000}{n\,d}$$

Where n = number of revolutions per minute;
and d = diameter of journal or pivot, in inches.

For crank pins and similar joints with alternating motion, the limiting bearing values given in the above formula may be doubled.

Roller Bearings.

Permissible Pressure, in Pounds per Linear Inch of Roller in Motion.

For cast iron	$p = 200d$
For steel castings	$p = 400d$
For axle steel	$p = 500d$
For tool steel	$p = 800d$
For hardened tool steel	$p = 1\,000d$

Where p = pressure per linear inch of roller;
and d = diameter of roller, in inches.

The foregoing values are for rollers and bearing surfaces of the same material; if rollers and bearing surfaces are of different materials, the lower value shall be used.

Ball Bearings. *Permissible Pressure on Balls of Hardened Tool Steel Running on Surfaces of the Same Material.*

For balls running on flat surfaces.....$P = 600d^2$
For balls running in grooves which have a radius of $\frac{2}{3}d$. $P = 1\,200d^2$

Where P = permissible load, in pounds per ball;
and d = diameter of ball, in inches.

Motors.

General. The kind of motor best adapted to any particular case depends upon local conditons, and should be left to the judgment of the engineer.

Hand-Power. If the bridge is operated by hand-power, the number of men and the time required to operate it shall be estimated on the assumption that the force one man can exert on a lever is 40 lb. with a speed of 160 ft. per min., developing about $\frac{1}{3}$ h.p. For calculating the strength of the machinery parts, the power of one man shall be assumed as 125 lb., but 150 lb. shall be the minimum used and applied to the extreme end of the hand-lever on any bridge.

Mechanical Power. If the bridge is operated by mechanical power, the motor shall be of ample capacity to move or turn the bridge at the required speed. All machinery parts shall be designed with sufficient strength to resist the greatest pressure which can be exerted by the motor, using the specified permissible strains. No matter what mechanical power is used, all bridges shall also be provided with hand-power operating machinery.

Hand-Brakes. Friction brakes, to be operated by hand or foot, shall be provided for all bascule and lift bridges, and for all swing bridges where the motor is located in the operator's house. They shall be attached to the secondary shaft of the motors which connect to the moving gear, and shall have sufficient capacity to stop or hold the moving span in any position under all conditions.

Safety Gates. In cities, all movable bridges, carrying highway traffic, which leave an opening in the floor when the bridge is open for navigation (as in swing bridges), shall be provided with safety gates. These gates shall be arranged so that the bridge cannot be opened before the gates are closed, and so that the gates cannot be opened before the bridge is closed and locked.

Operator's House.

Wherever mechanical power of any kind is to be used for operating a movable bridge, a suitable house shall be provided for the operator. The house shall be of such dimensions as required for the purpose for which it is to be used. It shall be placed in a position where the operator can observe the signals and see the approaching vessels and trains, and with enough windows of sufficient size so that his view will not be obstructed. If the operator's house is above or below the floor of the bridge, suitable steel or iron stairs with railings shall be provided to lead from the floor of the bridge to the floor of the operating house. The house shall be of fire-proof construction, consisting of a steel frame, steel floor-joists and a fire-proof floor. If the house contains motors and machinery, the floor shall preferably consist of steel plates, but, if the motors are located elsewhere, the floor between the joists may be of concrete construction. The sides and roof shall be of metal, concrete, or any other non-combustible material.

Heating.

Whenever the climatic conditions require it, provision shall be made for heating the operator's house. If steam power is used, the house shall be heated by a steam coil or radiator fed from the boiler. If electric power is used, the heat may be supplied by electricity. If gasoline is used, or any other power which cannot be utilized for heating, a coal, wood, petroleum, or gas stove, as directed by the engineer, shall be provided.

Steam Motor.

If a steam engine is used, it shall consist of a double-cylinder, reversing engine, the piston speed of which shall not exceed 200 ft. per min.; it shall develop the desired power and speed with a steam pressure of 50 lb. per sq. in. The engine shall be connected to the operating machinery by an approved friction clutch, arranged so that the moving and locking machinery can be operated alternately or stopped without stopping the engine.

Boiler.

The steam shall be generated by one or two upright, tubular boilers, each of which shall have twice the capacity of the engine. The boilers shall be designed for a steam pressure of 150 lb. per sq. in., and adapted to the kind of fuel specified by the engineer; they shall be of open-hearth steel, in accordance with the specifications for boiler plates appended hereto. They shall be cased in asbestos, covered with Russian iron.

Engine-Room.

The engine-room shall be provided with a steel water tank of sufficient capacity, a duplex, steam feed-pump, and an injector for each boiler, with necessary pipes and connections for feeding boilers separately or together, steam water-lifters with necessary strainers, flexible hose, and piping to lift the water from the river into the tank, a coal hoist and a steel coal-bin of sufficient capacity. The engine-room shall be provided with a suitable indicator for recording the positions of the moving span in turning and lock-

ing. A work bench with a full set of machinist's tools shall be provided, such as a vise, wrenches, chisels, hammers, files, oilers, oil-cans, and oil-tank. A whistle of suitable size shall be provided. All piping, where necessary, shall be covered with the best sectional covering.

Gasoline Motors.

If a gasoline motor or other internal-combustion motor is used, a low-speed engine of the most substantial kind shall be selected, the maximum piston speed of which shall not exceed 400 ft. per min. The engine shall have a reversing gear provided with approved friction clutches, to be operated by a hand-wheel. The countershaft connecting the engine with the operating machinery shall be provided with disengaging couplings, arranged so that the moving and locking machinery can be operated alternately and in either direction without stopping the engine. Motors of 10 h. p. and more shall be started by compressed air. The engine-room shall be provided with a water tank of sufficient capacity. The gasoline tank shall be located outside of the engine-house. The engine-room shall be provided with indicators for recording the positions of the moving span, and lifting, and locking apparatus. A work bench with a full set of machinist's tools, etc., shall be provided, the same as specified for steam motors.

Electric Motors.

If electricity is used as the motive power, the motors shall be of the railway type, series wound, single reduction, multipolar, water-proof, with steel frame and iron-clad armature of the required capacity. The capacity of each motor shall be estimated at normal speed and whatever voltage available according to the rating of the American Institute of Electrical Engineers, and be capable of carrying an over-load of $33\frac{1}{3}\%$ for 30 min. or 50% for 5 min. without injurious heating. The armature speed shall not be more than 600 rev. per min. With each motor shall be furnished a cut pinion and gear with a reduction such that the countershaft will make aproximately the required number of revolutions per minute (generally 100 to 125) under full-load rating. These pinions and gears shall be protected by a removable gear case. Motors connecting to vertical shafts should preferably have a vertical axis in order to avoid gearing.

All motors shall be of standard types in common use in order that extra parts may be readily procured, and shall be subject to the approval of the engineer.

Extra Parts.

The contractor shall furnish for each operating motor one extra armature, one extra field coil, one extra pinion, and one extra split gear. All such extra parts shall be fitted ready to place on the motors.

Controllers.

Controllers shall be placed in the operating house. Each motor or set of motors shall have one controller, that is, if two motors work

together, they shall be connected to one controller. They shall be of the reversible, series parallel type, fitted with interlocking, reversing cylinders, magnetic blow-out, etc., and shall be capable of varying and maintaining the speed of the motors from slow speed at the starting point to a maximum speed when full on, without sparking and without shock or jar. They shall be of ample carrying capacity to transmit for 30 min. 33⅓% more than the normal power required by the motors at full load, or transmit for 5 min. 50% more than the normal amount required by the motors, without injurious heating.

Resistance. Suitable resistances shall be furnished, so that the motors will start from a stand-still and attain full speed without causing sparking at the commutators of the motors, or without shock or jar to the bridge.

Cables. Unless the current supply is taken from more than one source, it shall be conducted to the switch-board in two independent conductors: one for the supply and one for the return current. These conductors, if the power has to cross a channel, shall consist of steel-armored submarine cables. Each cable shall be of sufficient capacity to carry safely the necessary current to operate the bridge with full over-load on the motors as herein specified. Each cable shall be composed of nineteen strands of tinned copper wire of not less than 98% conductivity, insulating walls of rubber not less than $\frac{5}{32}$ in. thick containing not less than 30% of pure Para rubber, one winding of tape, a lead sheath $\frac{3}{32}$ in. thick containing 3% of tin alloy, a substantial jute and asphalt serving, and an armor of galvanized-steel wire of suitable size for the diameter of the cable. The cables shall show, at 60° fahr., an insulating resistance of 500 megohms per mile after 5 min. electrification.

In the case of swing bridges, the cables shall be brought up, on or through the center pivot, with collector rings for the purpose of conducting the current to the controlling apparatus while the bridge is swinging. The collector rings shall be protected by a removable metallic casing.

Wiring. All wiring between the switch-boards, motors, and lights shall be of the best grade of rubber-covered, double-braided copper wire, and put up securely on porcelain insulators. No wires smaller than No. 14, B. & S. gauge, shall be used. All wires shall be drawn into place free from mechanical injuries, in loricated iron conduits located and arranged so as to be easily accessible for examination and repairs. All wiring shall be of sufficient capacity to carry safely the current required, with the over-load specified, without injurious heating.

Grounds. All ground connections to the structure shall be made with proper soldered terminals secured to copper plates of ample area

fastened in contact with the structural work, or in other manner satisfactory to the engineer. Care must be taken to locate the connections so that there will be ample metal and proper circuits to return the currents without damage to the structure.

Cut-Outs. An automatic circuit breaker, equal to the I. T. E. standard switch-board type, shall be placed on the motor circuit on each switch-board. Each cable, each line of motors, and each line of lighting, signal, indicator or other circuit shall be protected by suitable fuses of a pattern approved by the engineer.

Meters. One 600-volt meter with 5-volt graduation, and one 300-ampere shunt ammeter with 2-ampere graduation, both equal in quality to the Western type "F," shall be placed on each switch-board.

Switches. A suitable switch, of quick-break, railway type, shall be provided for each motor circuit and for each supply wire. All switches shall have ample carrying capacity for their respective loads with the specified over-load, but no switch shall have a capacity of less than 50 amperes.

Switch-Boards. There shall be placed in each operator's house a marble switch-board large enough to allow all necessary meters, switches, circuit breakers, fuses, etc., to be located thereon without crowding, so that each device can be reached and operated quickly and safely by the bridge tender. All switches, fuses, signal circuit, and submarine cable terminals shall be suitably named and labeled with neat metal plates, in accordance with their purpose and use. Switch-boards shall be located behind the controllers and convenient thereto, and placed at an angle of 45° with the side of the house. They shall be mounted on substantial iron supports and be thoroughly braced to the wall.

Electrical Indicators. An electrical indicator of approved design shall be placed in front of the controller, recording the movements of the bridge and the lifting and locking apparatus.

Contacts. The contacts or electrical devices for making or breaking the electric circuits to operate the electric indicators, or similar connections, shall be substantial in construction, reliable in action, completely protected from the weather, and shall be submitted to the engineer for approval.

Lights. In each operator's house shall be placed ten 16-c.p. lights, and additional lights about the machinery and such other points as the engineer may direct. Lights placed outside shall have water-proof sockets. Each set of lights shall be controlled by a switch on the switch-board in the operator's house.

Telephone. In each operator's house shall be placed a telephone, complete, with receiver, transmitter, battery, and magneto bell, and also a push-button and signal bell connected by the cable.

Signals. Suitable semaphores and signal lights shall be provided, as directed by the engineer.

All the electrical equipment, apparatus, etc., shall be of the most substantial character and of good finish, and all material and workmanship shall be subject to the approval of the engineer. The contractor, before beginning work, shall submit for approval complete plans, with details and specifications, showing the wiring and the parts he proposes to install. The working of the machinery and its efficiency to operate the bridge shall be tested to prove that it fulfills the specified requirements.

SPECIFICATIONS FOR SPECIAL METALS USED FOR MACHINERY PARTS.

Steel Castings.

Process of Manufacture.

1.—Steel for castings may be made by the open-hearth or crucible process. All castings shall be annealed unless otherwise specified.

Chemical Properties.

2.—Phosphorus ..... 0.05% maximum.
Sulphur ..... 0.05% maximum.

Tensile Tests.

3.—Minimum physical qualities as determined on a standard test specimen of ½ in. diameter and 2 in. gauged length:

Tensile strength, in pounds per square inch....	70 000
Elongation: percentage in 2 in.....	18
Contraction of area: percentage.....	25

Drop Test.

4.—A test to destruction may be substituted for the tensile test, in the case of small or unimportant castings, by selecting three castings from a lot. This test shall show the material to be ductile, free from injurious defects, and suitable for the purpose intended. A lot shall consist of all castings from the same melt or blow, annealed in the same furnace charge.

Percussive Test.

5.—Large castings shall be suspended and hammered all over. No cracks, flaws, defects, or weakness shall appear after such treatment.

Bending Test.

6.—A specimen (1 in. by ½ in.) shall bend, cold, around a diameter of 1 in., through an angle of 90°, without fracture on the outside of the bent portion.

Number and Location of Specimens.

7.—The number of standard test specimens shall depend upon the character and importance of the castings. A test piece shall be cut, cold, from a coupon to be moulded and cast on some portion of one or more castings from each melt or blow, or from the sink-heads (in case heads of sufficient size are used). The coupon or sink-head must receive the same treatment as the casting or castings, before the specimen is cut out, and before the coupon or sink-head is removed from the casting.

Sample for Chemical Analysis.

8.—Turnings from the tensile specimen, or drillings from the bending specimen, or drillings from the small test ingot, if pre-

ferred by the inspector, shall be used to determine whether or not the steel is within the limits in phosphorus and sulphur specified in Paragraph 2.

Finish. 9.—Castings shall be true to pattern, free from blemishes, flaws or shrinkage cracks. Bearing surfaces shall be solid, and no porosity shall be allowed in positions where the resistance and value of the casting for the purpose intended will be seriously affected thereby.

Steel Forgings.

Manufacture. 1.—Steel forgings may be made by the open-hearth or crucible process.

Chemical Properties. 2.—Phosphorus ..... 0.04% maximum.
Sulphur ..... 0.05% maximum.

Physical Properties. 3.—Minimum physical properties as determined on a standard turned test specimen of ½ in. diameter and 2 in. gauged length:

Tensile strength, in pounds per square inch... 55 000 to 65 000
Elongation: percentage in 2 in..... 28

Bending Test. 4.—A specimen (1 in. by ½ in.) shall bend, cold, 180°, around a diameter of ½ in., without fracture on the outside of the bent portion. The bending may be effected by pressure or by blows.

Number and Location of Tensile Specimens. 5.—The number and location of the test specimens to be taken from a melt, blow, or forging shall depend upon their character and importance, and, therefore, must be regulated by individual cases. The test specimen shall be cut, cold, from the forging, or full-sized prolongation of the same, parallel to the axis of the forging and half way between the center and the outside; the specimens shall be longitudinal, *i. e.,* the length of the specimen shall correspond with the direction in which the metal is most drawn out or worked. When forgings have large ends or collars, the test specimens shall be taken from a prolongation of the same diameter or section as that of the forging back of the large end or collar. In the case of hollow shafting, either forged or bored, the specimen shall be taken within the finished section prolonged, halfway between the inner and outer surfaces of the wall of the forging.

Sample for Chemical Analysis. 6.—Turnings from the tensile specimen, or drillings from the bending specimen, or drillings from the small test ingot, if preferred by the inspector, shall be used to determine whether or not the steel is within the limits in chemical composition specified in Paragraph 2.

Finish. 7.—Forgings shall be free from cracks, flaws, seams, or other injurious imperfections, and shall conform to the dimensions shown on the drawings furnished by the purchaser, and shall be made and finished in a workmanlike manner.

Annealing. 8.—All forgings shall be annealed.

Axle Steel.

1.—Axle steel may be made by the open-hearth or crucible process. Manufacture.

2.—Phosphorus ..... 0.05% maximum. Chemical Properties.
Sulphur ..... 0.05% maximum.

3.—Minimum physical properties, as determined on a standard turned test specimen of ½ in. diameter and 2 in. gauged length: Physical Properties.

Tensile strength, in pounds per square inch... 80 000
Elongation; percentage in 2 in..... 20

4.—A specimen (1 in. by ½ in.) shall bend, cold, 180°, around a diameter of 1½ in., without fracture on the outside of the bent portion. The bending test may be made by pressure or by blows. Bending Test.

5.—Turnings from the tensile test specimen, or drillings from the small test ingot, if preferred by the inspector, shall be used to determine whether the melt is within the limits in chemical composition specified in Paragraph 2. Sample for Chemical Analysis.

Boiler Plate.

1.—The steel used for boilers and fire-boxes shall be made by the open-hearth process. Manufacture.

2.—Phosphorus .....0.04% maximum. Chemical Properties.
Sulphur .....0.04% maximum.

3.—The physical properties required shall be as follows: Physical Properties.

Tensile strength, desired, in pounds per square inch 60 000
Elongation; minimum percentage in 8 in. $\frac{1\,500\,000}{\text{Ultimate strength.}}$
Character of fracture.....Silky.
Cold bends, without fracture.....180°, flat.

4.—The ultimate strength shall come within 4 000 lb. of that desired. Allowable Variations.

5.—Chemical determinations of the percentage of carbon, phosphorus, sulphur and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector. A check analysis shall be made from the finished material, if called for by the purchaser, in which case an excess of 25% above the required limits will be allowed. Chemical Analysis.

6.—Specimens for tensile and bending tests for plates shall be made by cutting coupons from the finished product, which shall Form of Specimens.

have both faces rolled, and both edges milled to the usual form of the standard test specimen, 1½ in. wide on a gauged length of at least 9 in.; or with both edges parallel.

Nickel-Steel.

Manufacture. 1.—Nickel-steel shall be made by the open-hearth process.

Chemical Properties. 2.—

	Plates, shapes and bars.	Rivets.
Phosphorus shall not exceed	0.04%	0.04%
Sulphur " " "	0.05%	0.04%
Nickel, not less than	3.00%	3.25%

Physical Properties. 3.—The physical properties required shall be as follows:

	Plates, shapes, bars and forgings. Pounds per square inch. Minimum.	Rivets.
Tensile strength	80 000	60 000 to 70 000
Elastic limit	50 000	40 000 min.

Elongation, percentage in 8 in., for plates, shapes, bars and forgings;

and also for rivets $= \frac{1\,600\,000}{\text{ult. strength}} =$ min.

Elongation, percentage in 2 in., for forgings = 25 per cent.

Bending Tests. 4.—Specimens cut from plates, shapes and bars shall bend, cold, 180°, around a diameter of three times their thickness, without fracture on the outside of the bent portion.

5.—Specimens cut from forgings (1 in. by ½ in.) shall bend, cold, 180°, around a diameter of 1 in., without fracture on the outside of the bent portion.

6.—Each rivet rod shall bend 180°, flat, on itself, without fracture on the outside of the bent portion.

7.—Rivet rods shall be tested as rolled.

Character of Fracture. 8.—The fracture of all tension tests shall show a fine silky texture, of a uniform bluish gray or dove color, free from black or brilliant specks, and shall show no sign of crystallization.

Annealing. 9.—All nickel-steel forgings shall be properly annealed.

Full-Sized Tests. 10.—Annealed eye-bars and similar members, when full-sized pieces are tested, shall comply with the following requirements:

Minimum ultimate tensile strength, in pounds per square inch..... 75 000.
Minimum elastic limit, in pounds per square inch, 45 000.
Minimum elongation in 10 ft., including fracture, 12%.
The fracture shall be mostly silky, and free from coarse crystals.
Full-sized pieces shall bend, cold, 180°, around a diameter of twice their thickness, without fracture.

Tool Steel.

1.—This steel is generally used for parts which require hardening or oil-tempering, such as pivots, friction rollers, ball-bearings and springs.

2.—Tool steel shall be made by the open-hearth or crucible process. Manufacture.

3.—Chemical Properties.

Carbon	1.00%	minimum.
Phosphorus	0.04%	maximum.
Sulphur	0.04%	"
Manganese	0.50%	"

Phosphor-Bronze.

Special phosphor-bronze shall be used for high pressures and slow speed.

1.—The metal shall have a minimum elastic limit in compression of 24 000 lb. per sq. in. Tests.

2.—A test piece shall be cut from a coupon to be moulded and cast on some portion of each casting. Test pieces shall be 1-in. cubes, finished. Test Pieces.

3.—Phosphor-bronze composed of the following ingredients and of the following proportions has given satisfactory results: Composition.

Copper	80%
Tin	15%
Phosphor-Tin	5%
Phosphorus	0.2%

Babbitt Metal.

1.—Babbitt metal composed of the following ingredients and of the following proportions has given satisfactory results and a low coefficient of friction (0.03 to 0.04): Composition.

Tin .....2 parts.
Zinc .....1 part.
Antimony, 5% of the weight of the tin and zinc.

2.—Melt the zinc and antimony separately. When both metals are melted, add the antimony to the zinc, and stir the mixture with an iron ladle. The tin is then added cold, and, when melted, the mixture is stirred thoroughly. Before pouring, a little sal-ammoniac is thrown on the surface to collect impurities. Method of Mixing.

DISCUSSION.

Mr. Dart. C. R. Dart, M. Am. Soc. C. E. (by letter).—For several years the writer has been connected with the construction of bascule bridges on the Chicago River, and, possibly, a few notes from his experience may be of interest.

The question of the best type of bascule bridge will not be discussed, the subject being too comprehensive for the time available and too dangerous to open. It will suffice to say that there are several types, patented and unpatented, and that each is the very best and cheapest. The variety to be adopted of any type (through, half-through or deck trusses, plate girders or riveted trusses, single or double leaf, etc.), must be decided in accordance with the conditions at the site. The single-leaf bridge is preferable to the two-leaf bridge, when practicable. Counterweight pits below the water level should be avoided, if possible, but when they are necessary, as, for instance, in the case of highway spans, the dirt from the roadway of the movable leaf should not be dumped into such pits.

Regarding the balancing of bascule leaves, the earlier trunnion bridges in Chicago were designed with the center of gravity in front of and above the trunnion, necessitating pneumatic buffers at each end of the travel. According to the later practice, the leaves are balanced as accurately as possible, so that, with no wind, they are stable in any position. Under these conditions, wooden bumpers are used, and have given no trouble.

The center lock now used for two-leaf spans is either a sliding bolt, moved by a motor working in one direction only, as suggested by the author, or consists of rigid tongues interlocking automatically as the leaves close, and having no movable parts.

One of the largest items of maintenance for bascule bridges carrying heavy highway traffic is the cost of floors on the movable leaves. The ideal wearing surface would be a 3 or 3½-in. creosoted wooden block pavement, if satisfactory means could be devised for securing such a pavement in place. Several designs have been submitted, but none has been considered worthy of trial. Where the traffic is light and the wear inappreciable, oak plank is probably the most suitable, as it will last for many years, when ventilated and drained properly. The quality of oak obtainable is rapidly growing poorer, however, and good white oak is becoming more expensive each year. The City of Chicago is using, with considerable success, a patented pavement consisting of 1-in. elm strips on edge, dipped in asphaltum and bolted together into sections containing from 5 to 10 sq. ft. This pavement wears well, and, thus far, has shown little tendency to decay.

The roadway of the fixed parts of a bridge, or of swing bridges, does not present such a difficult problem, as the engineer may choose from a number of well-tried pavements. Mr. Dart.

For the sidewalks on the movable leaves, 2 by 4-in. hardwood (maple), laid with $\frac{1}{4}$-in. to $\frac{3}{8}$-in. open joints, has been found satisfactory. Where navigation traffic is large, wooden sidewalk stringers should be preferred, for, when struck by a boat, the damage or distortion in the structure will be confined to the point of injury, and can be repaired by carpenters readily and quickly; with steel stringers, several panels of sidewalk may be pulled out of position, and the sidewalk brackets for some distance away may be bent.

Many specifications are lacking in definiteness as to the quality of the lumber desired. The old-time requirement, which has been copied and re-copied for years, specifies all heart lumber, and can no longer be filled. This fact should be recognized by naming a definite grade of lumber, which can be obtained on the market, in accordance with a stated well-known commercial inspection. This is but fair to the contractor, as it gives him a basis for estimating and for purchasing his material.

Regarding machinery, one of the writer's greatest troubles has been to obtain properly fitted keys. The impression seems to prevail, in machine shops generally, that a bridge engineer has no idea of the refinements of engine and machinery practice, and that almost any kind of workmanship is good enough for a bridge. It seems, also, that in many bridge shops it is absolutely impossible to obtain accurate work. The fitting of keys is always a slow and laborious task, running high in shop cost, and it will be slighted, if not carefully watched. Unless keys are accurately fitted for full side bearing, they are certain to work loose.

All machinery for bridges which are moved frequently should be inspected daily, and all keys should be tested, and driven tight when necessary. Neglect of this precaution may result in the loss of gears, and a tie-up of the bridge.

In the design of machinery, particular stress should be laid on the requirement of accessibility for inspection and repairs, with especial attention to the matter of space for driving and tightening the keys and for their withdrawal without the necessity of removing an entire shaft with its gears. It would hardly seem necessary to caution a designer on this point, yet it is one which is very often overlooked, and is an important factor in the expense of machinery maintenance.

In his paragraph on toothed gearing, the author requires that all cast pinions shall be shrouded. An alternative method of reinforcing pinion teeth is to increase the face, and, in most cases, the writer considers this method preferable, as cleaner castings can

Mr. Dart. be obtained, with a better opportunity for examining the teeth and chipping them, if found untrue. The teeth of slow-moving gears can also be strengthened by reducing their height, and, in the case of pinions which make only two or three revolutions during an operation of the bridge, the tooth height can be cut down to one-half the pitch, since the wear on such parts is inappreciable. Of course, this cannot be done on a twelve-tooth pinion, the minimum limit set by the author.

It is the writer's practice, and a practice which is, he believes, quite general, to increase the thickness of the teeth of pinions engaging racks at the expense of the rack teeth. A very large addition to the strength of the pinion teeth can be thus obtained and the rack teeth will still be the stronger of the two. Breakage should, of course, occur in the pinion—the part most easily replaced—rather than in the rack.

A minor consideration in connection with the machinery is the question of grease cups. Iron cups are generally preferable, as brass cups are a great temptation to thieves. The writer knows of no iron cup on the market, however, which is fully satisfactory. He has found it necessary to add a leather packing in order to prevent the escape of grease around the plunger of the cup.

As to electrical equipment: In the author's specifications only direct-current motors are considered, whereas it is often found that only alternating current is available. As existing alternating current motors are deficient in starting torque, they are not suitable for operating structures of some size, and it is to be hoped that a satisfactory motor will soon be designed, or an arrangement devised, for operating with these motors. The writer is informed that railroad draw-spans of moderate size are operated in this way at the present time, but, probably, in these cases, the operations are not frequent. When two motors are necessary, the difficulties are increased, as induction motors cannot be operated in series.

The question of automatic control for electrically-operated bascule bridges is a debatable one. It is practically impossible to make the control absolutely and reliably "fool-proof," as contacts for cut-outs and limit switches are continually getting out of order, especially if exposed more or less to the weather. If the contacts fail only once a year, such failure may result seriously. Any system of complete automatic control will lead to carelessness on the part of the operator, and a failure of any of the complicated parts of such control to act might cause an accident. If the control is designed so that the bridge will not operate in case of failure, the structure may be tied up at the busiest time of the day, with a possibility of damage suits on account of delays to traffic. The writer, therefore, does not advocate full automatic control, but pre-

fers a simple system of limit switches which will act only when the safety points are passed, and, having acted, require some extra work on the part of the operator to close the circuits. Mr. Dart.

For bascule bridges with two leaves, it is a wise provision to have the submarine positive-power cables in duplicate, but with only one return cable. Of course, it is also advisable, where possible, to have two independent sources of power, with the wiring arranged so that both leaves can be operated with current from either source. The not infrequent interruptions of power in some localities often make such an arrangement imperative.

In specifying cables, the exact size required should be stated, as there seems to be a wide difference in contractors' figures as to their carrying capacity. The least that should be done is to mention some definite code or wire tables, but an exact specification of size will save many disputes between the contractor and the engineer.

In the paragraph referring to safety gates, such gates are called for in bridges which leave an opening in the floor when the bridge is open for navigation. In cities with heavy street traffic across a bridge, gates should always be provided for the purpose of stopping the traffic, and they should most certainly be provided where an opening in the floor occurs during the operation, even if such opening is closed by the leaf when in its extreme raised position. Gates should be arranged so that each one can be operated independently of the other, and in any order desired, and in no case should they be operated automatically by the movement of the bridge.

The writer desires to give credit to the Bridge Department of the City of Chicago for many of the ideas here advanced, as it is through the operation of the bridges by the city that such ideas have been developed.

J. R. Worcester, M. Am. Soc. C. E. (by letter).—This paper is so comprehensive that it would be surprising if a few inaccuracies had not crept in, and it is to be expected that there will be many differences of opinion in minor particulars. It is much easier to criticise than to construct, and the presentation of such a complete treatise on the subject affords the Society an invaluable opportunity to collate all the divergent ideas of its members. With this end in view, the writer submits the following suggestions: Mr. Worcester.

In speaking of jack-knife draws, the author says: "When the bridge is closed, one end of each girder rests on the shoe at the abutment, while the other end rests on the pivot." This is not usually the way these bridges have been built in the vicinity of Boston, where many have been used for the last half century. Although the type has the manifest objection that no ties can be used

Mr. Worcester. under the rail, it has been used so extensively, and—as far as the writer knows—with no accident due to this lack of ties, that, perhaps, it deserves a more complete description.

The regular form consists of a deck truss under each rail, one or more needle-beams under the trusses near the free end, and a gallows-frame located over the pivots, braced and anchored back on the shore, to which the ends of the needle-beam are connected by cables. The trusses are generally in the form of wooden Howe trusses with unusually wide chords, but, in a few instances, steel Warren trusses have been used. The trusses are of unequal lengths, with the pivots at different distances from the face of the abutment, so that, when the bridge is opened, the trusses will lie side by side over the abutment, the shortest being next to the abutment and the longest against the parapet. When the bridge is closed, the trusses are arranged to bear not only upon the pivot and the opposite abutment, but intermediate support is obtained at the face of the abutment upon which the pivots are located.

The needle-beams are hinged to the trusses by vertical pins at equal distances from the pivots, and hence are not parallel to the abutments. They are long enough so that the cables attached to their ends will give a proper clearance for the train, and strong enough to carry the dead weight of the bridge.

The cables at their upper ends are connected to the short ends of the levers hinged on the gallows-frame, and arranged so that, by pulling down the long end, the bridge is raised from the abutments. The connections between the cables and the levers are in the same vertical plane with the pivots, and spaced so that the cables lie in vertical planes. In fact, all the vertical hinge joints are carefully placed to produce parallel motions of all parts. The most difficult parts to arrange for this parallel motion are the ties between the trusses. These must be hinged on the center line of the truss, and, in order to allow the necessary swing for the ties so that the trusses may be closed up together, the ties have to be made with bent ends—**S**-shaped—which, of course, seriously affects their strength.

The bridges are usually operated by a straight rack attached to an arm, hinged to the bottom chord of the longest truss and carried past a pinion on a vertical axis placed on the abutment.

The writer cannot agree with the author's recommendation, in the paragraph on "Rack and Track," that in any case the rack and track "shall preferably be cast in one piece." The rack is much more likely to be injured accidentally than the track, and should always be arranged so that a segment can be replaced without disturbing the track. Should not the specifications also provide that the patterns for special gearing, such as racks, should be furnished with the bridge, so that broken parts can be easily replaced?

An important provision with regard to bascule bridges when in two leaves, is that the ends shall be provided with latches of sufficient strength to transmit enough shear from either leaf to the other in order to equalize the deflection under any condition of loading. This provision might well be introduced in the paragraph on "Bascule and Lift Bridges." Mr. Worcester.

ALBERT HENRY SMITH, ASSOC. M. AM. SOC. C. E. (by letter).— The presentation of this interesting subject by Mr. Schneider will, no doubt, be fruitful of much additional information. Mr. Smith.

It is a pleasure to read a set of specifications which covers the subject so completely and in such a satisfactory manner. The writer respectfully wishes to confirm and add to some of the data presented, and will class his remarks under the several headings given in the paper.

Selection of Design.—The writer believes that, like many another new idea, the bascule idea is and has been overworked, and that such bridges are frequently built where they are not needed, thus causing an excessive first cost and high operating charges.

In addition to the three conditions mentioned by the author as precluding the use of a swing bridge, the following two conditions should cause the consideration of a bascule bridge:

1.—If the protection pier of a proposed swing bridge will render valuable dock property useless;

2.—If the bridge is to be located where the traffic over it is very heavy, and the bridge has to be opened very often, say more than twenty-five times a day.

This second condition may be very important, on account of the time saved in operating a bascule as compared with a swing.

Rim versus Center-Bearing Types.—The points in favor of the center-bearing bridge are all well taken, and are borne out by the writer's experience.

Until recently, the bridge designers of the Middle States have given the rim-bearing type the preference. Ten years ago, in Cleveland, Toledo and Chicago, there were very few center-bearing bridges. Since that date, in addition to the rim-bearing type, the bridges built in these places have been of the center-bearing, combined rim and center-bearing, and bascule types.

The only objectionable member in a center-bearing bridge, from the shop superintendent's point of view, is the center or pivot casting. This, for bridges approaching 1 000 tons in weight, becomes quite large in diameter, and somewhat difficult to machine.

In the rim-bearing type, if designed in accordance with good practice, it usually works out that the number of rollers required is about as many as can be placed under the drum.

Mr. Smith. The matter of using plenty of rollers is an important one. In very few bridges of this type do all the rollers carry their proper share of the load, and the writer has seen many of these bridges in which not more than two-thirds of the rollers were carrying any load, the remainder not being in contact with the drum at all, and sometimes three or four consecutive rollers being entirely free.

It is a very difficult matter to get truly conical surfaces for faces or treads for the drum and the track, and equally difficult to get the bottom track properly set in the field. In such cases, everything is in favor of the single pivot; it is machined as a unit and erected as a unit, and there is but one place for the load to go.

The working loose of center pivots of purely rim-bearing swing bridges is quite common. Of the two methods suggested for preventing this, the writer prefers the second—that of rigidly connecting the center to the circular track by radial struts. This not only assures a rigid pedestal, but is an aid to the erection in getting the track circular and concentric with the center pivot.

The combined rim and center-bearing type costs more than the purely rim-bearing type, due to the increased cost of the pivot, radial girders and transverse girders, and, aside from the point in its favor of fixing the center pivot, has nothing in particular to commend it. The detail suggested for the live-ring is a good one.

Of the thousand and one devices used for end lifts, the wedge is by far the most reasonable and satisfactory. The proper shaping of the wedge and its seat provides a most satisfactory means of bringing the bridge into alignment and keeping it there, independent of the latch-pin.

Power Required to Operate Movable Bridges.—For mechanically-operated bridges, a safe rule for the amount of power is to allow 1 h. p. for each 15 tons of weight to be swung. This provides ample margin for taking care of excessive wind pressures, and gives plenty of power to open and close the bridge rapidly. For bridges of 300-ft. span, the gear ratios should be such that the bridge can be opened in 1 min. from the time the ends are unlatched and released.

The foregoing rule has been checked repeatedly by ammeter readings taken on electrically-operated bridges, the swinging weights of which have varied from 300 to 1 000 tons. Readings on these bridges show a power reserve sufficient for all possible contingencies. It is usually desirable to open and close a bridge with as little delay as possible, and this can only be done by having ample power to furnish a heavy starting torque.

Mr. Smith.

The effect of wind pressure on an equal-armed swing bridge has often been observed by the writer. Both observations and tests confirm Mr. Schneider's statement, that it may take twice as much power to operate a bridge on a windy day as in calm weather.

Concerning the use of a brake on electrically-operated bridges, the writer has always made this a part of the equipment, and he has noted that the bridge operator always uses it to check and control the speed of the bridge when the current is cut out, it being a great aid in stopping the bridge exactly at the latching point. This brake is attached to a shaft, as close to the motor as possible, and is arranged so that it is applied by means of an adjustable weight which is controlled by a foot-lever, the object being to prevent the operator from setting up the brake so hard as to wreck the machinery. The connection between the foot-lever and the brake weight can usually be made by the use of flexible wire rope and pulleys.

The writer prefers the electric motor to the steam or gas engine, if a reliable source of electric current is obtainable. If possible, current should be brought to the switch-board from two independent lines or sources, so that in case one line is dead, the other will be available, the chances of both lines being dead at the same time being very small.

If the bridge is operated very frequently, the electric equipment should be in duplicate. Each motor should be operated by its own controller, or both motors by one series parallel controller, each motor being of sufficient capacity to handle the bridge independently of the other. With this arrangement, the possibility of not being able to operate the bridge at any time is very remote.

The writer has in mind an electrically-operated bridge with but one source of electric current. The current failed just as the operator was opening the bridge, and the bridge and piers were damaged seriously by the approaching vessel. It was at first proposed to remedy this by installing on the bridge a storage battery of sufficient capacity to operate it three or four times. This was not done, but connection was made to an independent line.

One point of design is frequently overlooked, or not properly taken care of, and that is the brackets and bearings for the rack pinion shaft. Usually, these brackets and bearings are made entirely too light. An examination of about 40 swing bridges, a few years ago, showed that repairs and reinforcement had been made at this point on nearly every one. This is particularly true of power-operated bridges. These brackets and bearings should be designed, not simply strong enough to withstand the strain due to opening the bridge, but to withstand the maximum possible torque

Mr. Smith. that the motor, engine, or brake can develop. The brackets should preferably be steel castings, cast in one piece.

For bridges operated by hand power, it usually develops that one man, using a force of 40 lb. on the hand-lever and moving at the rate of 200 ft. per min., can swing 150 tons of bridge through 90° in 5 min. Hand-operated bridges rarely weigh more than 300 or 400 tons, and two men can open such a bridge in 5 or 6 min. Tests made on rim-bearing bridges of the poorest design confirm the statement that this amount of power is ample.

Concerning the amount of power to be assumed in calculating the strength of gearing, the writer would suggest that, in addition to the power of one man being taken at 125 lb., one should multiply that weight by a number equal to as many men as it is possible to place at the hand-lever.

The writer has seen pinions, amply strong enough to swing the bridge, have their teeth all stripped from them when the first attempt was made to open the bridge in the spring. The bridge would not move, and additional men were asked to lend a hand until something did move.

Materials.—It is noted that cast steel may be used for wedge bearings, and that steel on steel for sliding surfaces is not to be desired. Has anyone ever had any trouble with cast iron sliding on cast steel, when used for wedge surfaces?

It is noted that Mr. Schneider has a strong objection to the use of bevel gearing, and the writer hopes that additional information will be given. He has never had any trouble with cast bevel gears, except where the ratio between the pinion and gear was unusually large.

Mr. Hess. H. D. Hess,* Esq. (by letter).—The very general acceptance of Mr. Wilfred Lewis' discussion of the strength of gear teeth† would make the adoption of his formula seem desirable in specifications like Mr. Schneider's.

Bach's formula applies correctly only to the discussion of pinions with from 10 to 12 teeth, while Mr. Lewis' formula applies, with equal accuracy, to gears with any number of teeth. Where the gear ratio is considerable, and an analysis of the pinion tooth only is required, Bach's formula is all that can be desired; on the other hand, when an analysis of the gear tooth is required, as, for instance, with a shrouded pinion, or where the pinion and the gear are of different materials, or where the pinion has a large number of teeth, then Bach's formula fails.

Mr. Lewis' formula, when applied to 15° involute teeth, is of the same form as that given in the specifications, except that

* Assistant Professor of Machine Design, Cornell University.

† *Proceedings*, Engineers' Club of Philadelphia, 1893, p. 116.

$y = \left(0.124 - \frac{0.684}{n}\right)$, where n is the number of teeth on the pinion or gear under consideration. Mr. Hess.

The fiber stress is reduced as the speed of the gears increases; Mr. Lewis recommends that the fiber stress be reduced according to the formula, $\left(\frac{600}{600 + V}\right)$, where V is the linear velocity, at the pitch circle, in feet per minute. This is in close accord with the factor given in the specifications, $\frac{10}{\sqrt{V}}$.

In general, for "cut" gears, the pressure can be assumed as distributed over the entire width of the tooth. Cast gears and "cut" gears in which the support is likely to be deflected sufficiently to cause the tooth pressure to act at the corners, should be limited to a face width of from one and one-half to two times the circular pitch.

J. E. Greiner, M. Am. Soc. C. E. (by letter).—The writer recently had occasion to make some tests on the power required to operate a double-track swing bridge, and submits herewith the data and results as a contribution to the discussion of Mr. Schneider's most excellent paper. Mr. Greiner.

The bridge tested was the Baltimore and Ohio Railroad double-track swing bridge crossing the Calumet River, at Chicago, Ill.

Total length	250 ft.
Width, from center to center of trusses.	28 "
Angle	90 degrees.
Pitch diameter of rack	34 ft. 2 in.
Center pivot—diameter of disk	2 ft. 0 in.
Total load on pivot—bridge swinging	1 240 000 lb.
Operating Power	Two 50 h. p. General Electric motors, Class 57; efficiency, 85%; rated speed, 470 rev. per min.; 500 volts, direct-current.

The bridge revolves on a center pivot which carries the entire dead load. When under traffic, the load on the center pivot is partially relieved by a wedge arrangement which supports the center cross-girders. The ends of the bridge are raised by jacks instead of wedges, these jacks being operated by toggles arranged so as to convert a horizontal motion to a direct vertical lift.

The tests, which were personally conducted by W. R. Edwards, Assoc. M. Am. Soc. C. E., were made on February 20th and 21st, 1907, the temperature ranging from 18 to 35° fahr. The results of these tests, as given herein, are averages of from four to six

Mr. Greiner. operations, and show the indicated horse-power determined by simultaneous readings of the volt meter and ammeter. The entire current passed through the ammeter before being divided for the two motors, so that this reading gives the indicated horse-power, whether one motor or both were used. As many readings as possible were taken during each operation, and the results of these readings were averaged. The readings were taken in swinging the bridge through an arc of 90°, from the instant of starting to the instant of stopping.

The following is the time required:

Acceleration	20 sec.
Uniform movement	40 "
Retardation	15 "

The acceleration took place almost uniformly in an arc of 10°; the uniform motion was through an arc of almost 70°; the retardation took place in from 5 to 10 degrees.

The average indicated horse-power required to swing the bridge, open and closed, was:

During acceleration	44.9
During uniform motion	35.6

The retardation was accomplished by shutting off the current and applying the brake.

As the efficiency of the type of motor used is about 85%, the average effective horse-power for swinging the bridge was:

During acceleration	38.17
During uniform motion	30.3

Mr. Gay. MARTIN GAY, M. AM. SOC. C. E. (by letter).—In the hope of contributing something useful on the subject of Mr. Schneider's very valuable paper, the writer feels moved to offer some observations from the point of view of an engineer who has been charged with maintaining, for some years, a number of heavy swing bridges.

He has been particularly impressed by the fact that economy in first cost adds greatly to the expense of maintenance—economy, not only in workmanship but in material—and he ventures to submit that, however praiseworthy efforts in that direction may be, and however great the pleasure an engineer may take in the nice proportioning of the members of a structure to their work, such efforts are misplaced in the design of a moving bridge. The more rigid and unyielding the base, as well as the moving parts, of a turn-table, the easier and cheaper will be its maintenance.

The proper preparation of the top of the pivot pier under the track is not an expensive matter. It can be bush-hammered to such

a level surface that the bed-plates will have practically perfect contact with the stone. The bed-plates should be planed on top and bottom to parallel surfaces, and both surfaces of the wheel treads should be machined accurately. Mr. Gay.

A heavy cover-plate on the lower flange of the drum, breaking joint with the flange angles and with the segments of the upper wheel tread, while offering some difficulties in construction, will add very much to the stiffness of the tread.

It is a very common practice to make up the joints in the base of a turn-table with shims, red lead, or some kind of cement, which will eventually work out. This should not be tolerated, as the necessity for it is evidence of bad workmanship which a proper shop inspection would have detected. The members should fit accurately, metal on stone and metal on metal, without the interposition of any filling.

As it is not in the skill of man to make an absolutely perfect machine, it is probable that no rim-bearing bridge has been built in which every wheel carries its full share of the load at all times, and it happens frequently that several wheels together carry no load. Therefore the drum should be very stiff. Of course, it will be as high as conditions permit, and it may be advisable to space the web stiffeners much closer than is usual in fixed plate girders. The drum should also be thoroughly braced against deformation, by struts radiating from the pivot.

Embedding the center casting in concrete does certainly interfere with its moving on its base, but, as Mr. Schneider remarks, "it does not prevent the pushing, pulling and wearing around the collar of the pivot," nor does it prevent the overturning of the casting, if the drum has a tendency to eccentric movement.

Of course, the center casting should be secured in place by bolts, concrete, or any other means available; but the best insurance against its movement, and the eccentric motion of the bridge, with attendant damage to the turn-table, is a substantial, well-designed live-ring system, of the best workmanship. It should be built on the lines indicated in the paper, and should be so well braced and rigid, that, if a wheel or series of wheels, through maladjustment or accident, tends to roll out of the true orbit, it will not be able to deform the ring, or leave its proper position on the track.

One of the most frequent, annoying, and sometimes expensive repairs is made necessary by the inadequate fastening of the pinion bracket to the drum. The bracket itself should be more massive than may appear necessary to the draftsman at his table, and, in the opinion of the writer, should be built up of structural shapes rather than cast from iron or steel. It should be strongly fastened, by rivets if possible, and further secured by long horizontal brackets attached to the lower flange of the drum. The pillow-blocks and

Mr. Gay. caps should be fastened with steel bolts, turned to a driving fit, in reamed holes. These remarks on the inadequate fastening of pinion brackets apply to center-bearing bridges also, the writer having had trouble of this nature with one recently built.

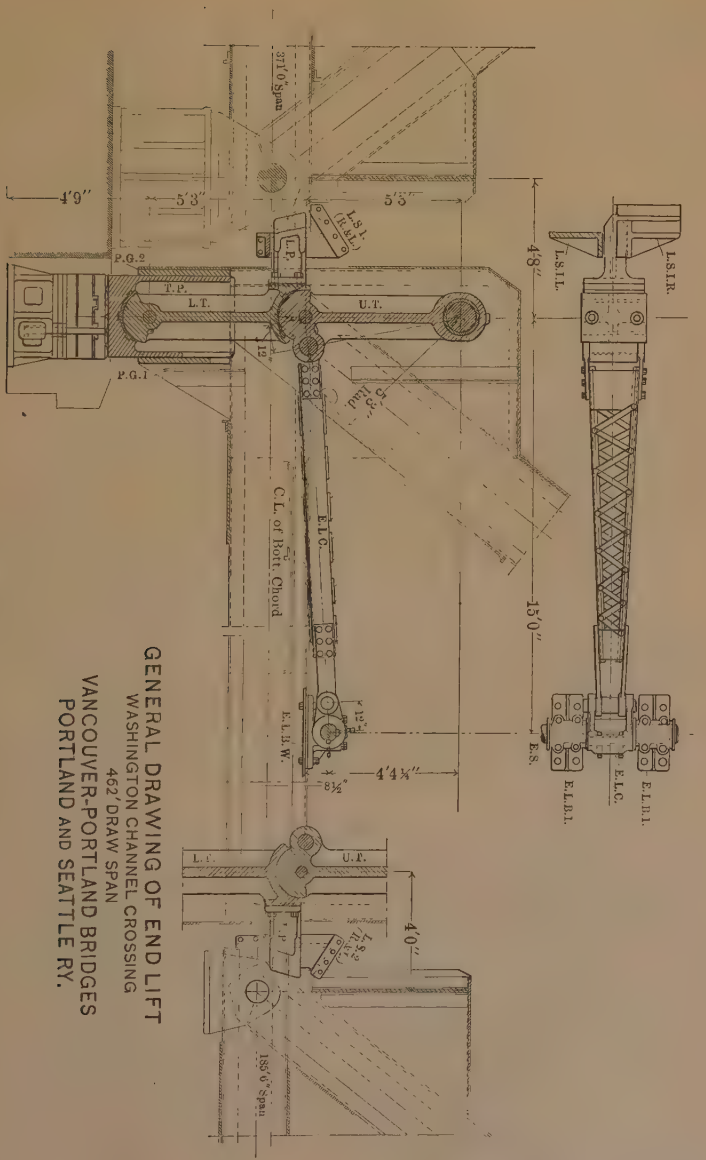
It is with diffidence that the writer ventures to disagree with Mr. Schneider even on the comparatively unimportant matter of shaft couplings. The arguments in favor of loose couplings are sound, and yet, on a number of bridges of varying lengths and construction, up to a pin-connected span of 400 ft., no trouble has been experienced with rigid flanged couplings; whereas, with claw couplings, there is some lost motion, there is usually a disagreeable chatter, and they have been known to fail. In fact, the writer usually keeps on hand a few flanged couplings with which to replace the claw couplings when necessary.

Another matter, quite unimportant until viewed as a very frequent item of repairs, is the securing of gears, couplings and collars to the shafting with keys and set-screws. What virtue there is in placing keys and set-screws at an angle of 120° with each other is not apparent, but, if there is any advantage in that position, it should be known, and will undoubtedly be adopted; but, until the superiority of 120° over 90° is made clear, it is probable that engineers will adhere to the old, well-established practice, sanctioned by experience.

Mr. Modjeski. RALPH MODJESKI, M. AM. SOC. C. E. (by letter).—No subject of equal importance has received less attention in the way of publications and discussion in professional papers than the one treated by the author. In all the so-called standard specifications for bridge superstructures clauses relating to movable bridges seem to have been carefully avoided, and yet such bridges constitute the most expensive part of a railroad, both as to first cost per foot of track and as to maintenance and operation. The subject is quite complex, covering a great variety of structures under variable conditions. Mr. Schneider's very clear and concise paper is valuable because it embodies, probably, the first general specification on movable bridges, and because it opens the field for a thorough discussion.

With regard to end lifts, the author expresses his preference for wedges, and does not recommend "that kind of end lift which supports the ends of the bridge, when closed, on rollers, toggle joints or links." The writer has used rollers quite successfully. Plate XXXI shows the roller end lifts used by him in the Rock Island Railroad and Highway Bridge which have given entire satisfaction. It will be noticed that the lower casting has inclined instead of horizontal bearing surfaces; and that each roller, when the span is fully raised, comes into contact with a convex surface

Mr. Modjeski.



GENERAL DRAWING OF END LIFT
WASHINGTON CHANNEL CROSSING
462' DRAW SPAN
VANCOUVER-PORTLAND BRIDGES
PORTLAND AND SEATTLE RY.

Fig. 1.

Mr. Modjeski.

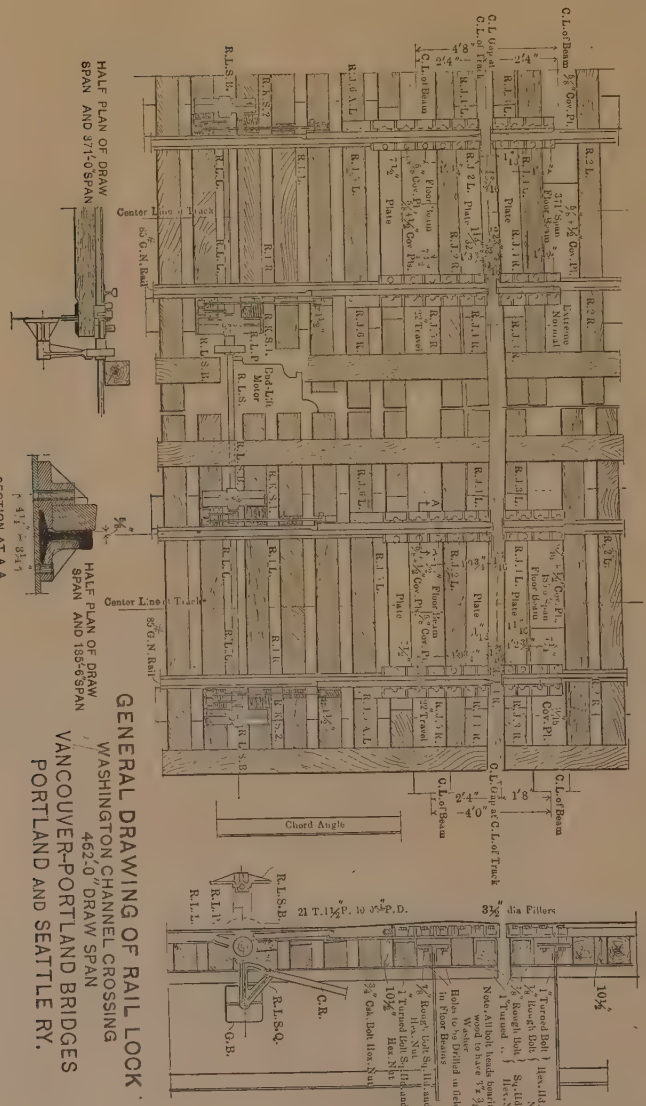
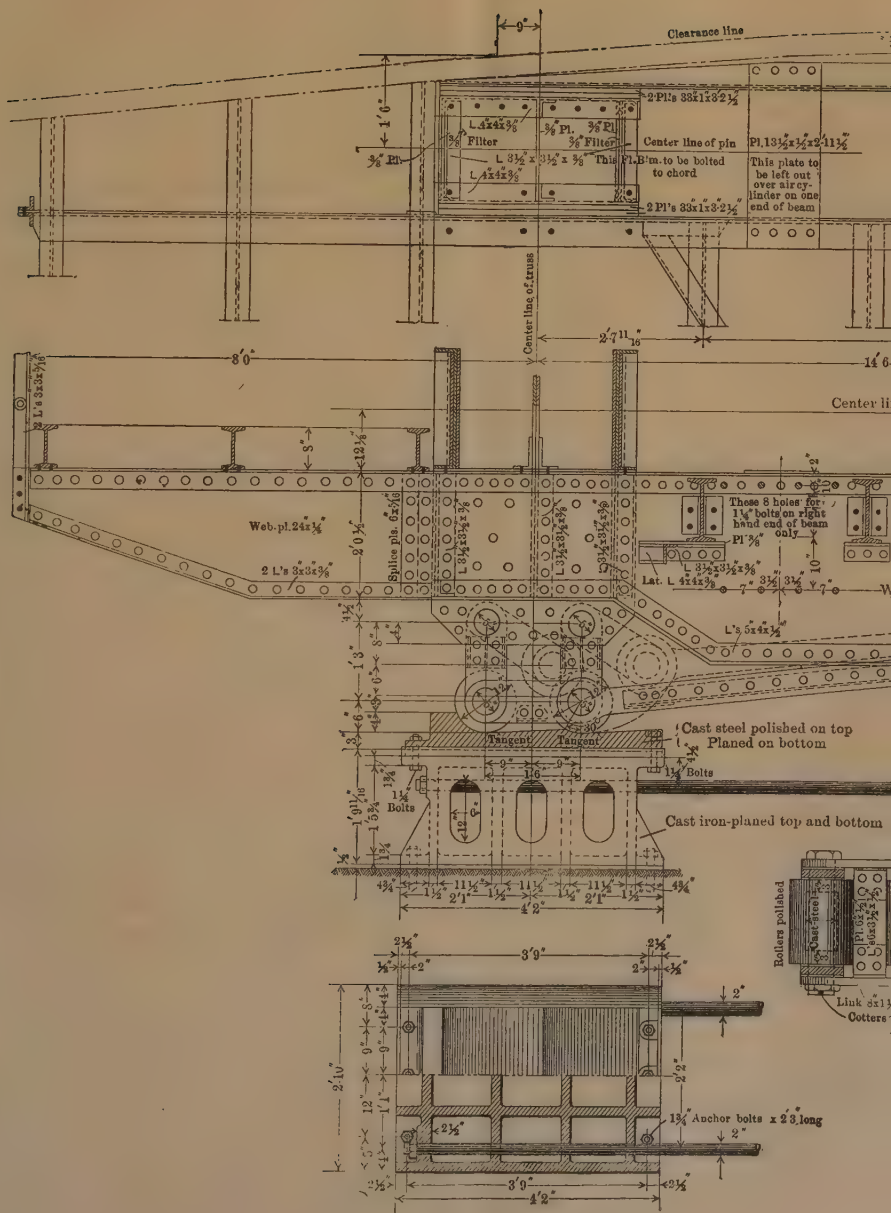
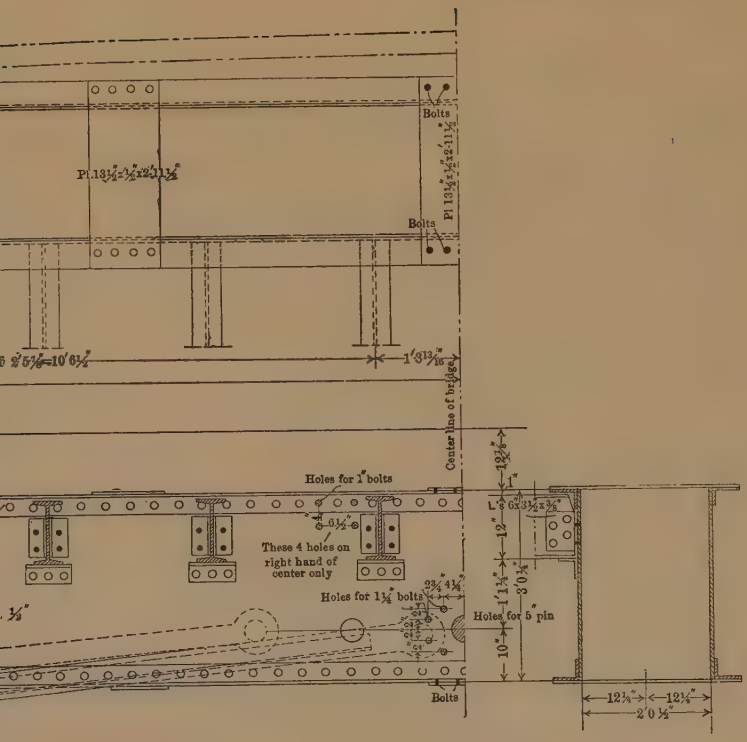
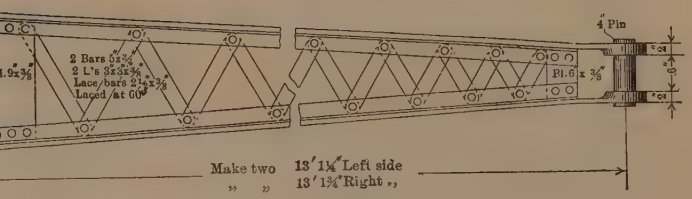


Fig. 2.



Clearance line
2 Pl's 33x1x3-2½
L 4x4x⅜
⅜ Filter
⅜ Pl.
⅜ Pl.
⅜ Filter
Center line of pin
Pl. 13½x½x2-11½
L 3½ x 3½ x ⅜
This Pl. B'm. to be bolted to chord
This plate to be left out over air cylinder on one end of beam
L 4x4x⅜
2 Pl's 33x1x3-2½
Center line of truss
2'-7 11/16"
8'0"
14'6
Center li
Web. pl. 24x¼
2 L's 3x3x⅜
Splice pls. 6x5/16
These 8 holes for 1¼" bolts on right hand end of beam only
Pl ⅜
Lat. L 4x4x⅜
L's 5x4x½
Tangent
Tangent
Cast steel polished on top
Planed on bottom
1¼ Bolts
1¼ Bolts
Cast iron-planed top and bottom
Rollers polished
Cast steel
Link 8x1
Cotters
1¾" Anchor bolts x 2'3 long
3'9"
4'2"
2'10"





ROCK-ISLAND BRIDGE
365'7" DRAW SPAN.
S OF END-LIFT, FLOOR BEAM AND CAST BEARINGS.

which acts as a stop. By this arrangement, when the end of the span is fully raised, the contact between each roller and the bed-plate is an area, and not a line as is the case in many roller lift designs. This device has the further advantage of centering the span and locking it automatically; and there is less frictional resistance than when wedges are used. Owing to the sloping bearing surfaces of the bed-plates, the rollers tend to remain in place when set. Mr. Modjeski.

The writer presents, in Fig. 1, a design for an end lift of an entirely different character, which will be used for a 462-ft., double-track span over the Columbia River, and also for a 521-ft., double-track span over the Willamette River. The toggle has been made very powerful and substantial, and suitable for heavy loads. The bearing takes place on a nest of segmental expansion rollers which right themselves whenever the bearing blocks are raised and the pressure is removed. As this design has not yet been installed, and, therefore, has not received the sanction of a trial, the writer will abstain from discussing it herein. Both these devices dispense with the automatic latches required by Mr. Schneider's specifications. It has been the writer's experience that such latches, if effective in holding the bridge truly lined up, must have very little play when in a locked position, and, therefore, must always produce a certain shock in locking, and this the writer believes to be objectionable in heavy bridges.

The author calls for rail-lifts in his specification. The writer believes that the rail-lift will soon be entirely superseded by safer devices; one of them recently recommended for adoption by, at least, one important railway system, is essentially the same as the rail-lock used by the writer, first on the Rock Island draw, and, later, on all draw spans designed by him.

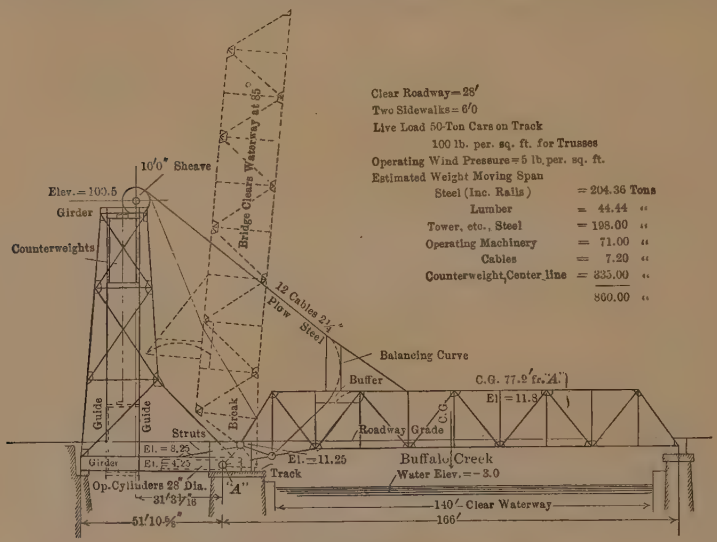
Fig. 2 shows the rail-lock which will be used on the Columbia and Willamette River draw spans. The principal advantages are: Practical continuity of the rail over the gap—the key is shaped so that it raises the wheel slightly before it reaches the end of the gap, and sets it down on the second rail some distance from its end; and the rails are permanently fastened to the ties.

A sliding-key rail-lock was used on the old Rock Island draw span before its reconstruction. In the writer's design, the old form of lock has been modified by making the key beveled lengthwise and crosswise on top, instead of level, and by providing more efficient guide-bars.

Wilbur J. Watson, M. Am. Soc. C. E. (by letter).—The writer has recently been called upon to design several bascule bridges, and will take advantage of the author's invitation to discuss this feature of the paper. Mr. Watson.

Mr. Watson.

The bridge illustrated in Fig. 3 is a single-leaf highway span, 166 ft. from center to center of bearings, now under construction at Buffalo, N. Y. In this design, the balance is maintained by a detached counterweight, moving vertically between fixed guides attached to a tower which carries two 10-ft. cast-steel sheaves, over which pass twelve 2¼-in. plow-steel cables, which pass around grooved tracks attached to the trusses, and placed so that the lever arm of the ropes about the point of rotation maintains a constant ratio to the lever arm of the center of gravity of the moving span. The counterbalancing scheme of this bridge, as well as the hydraulic



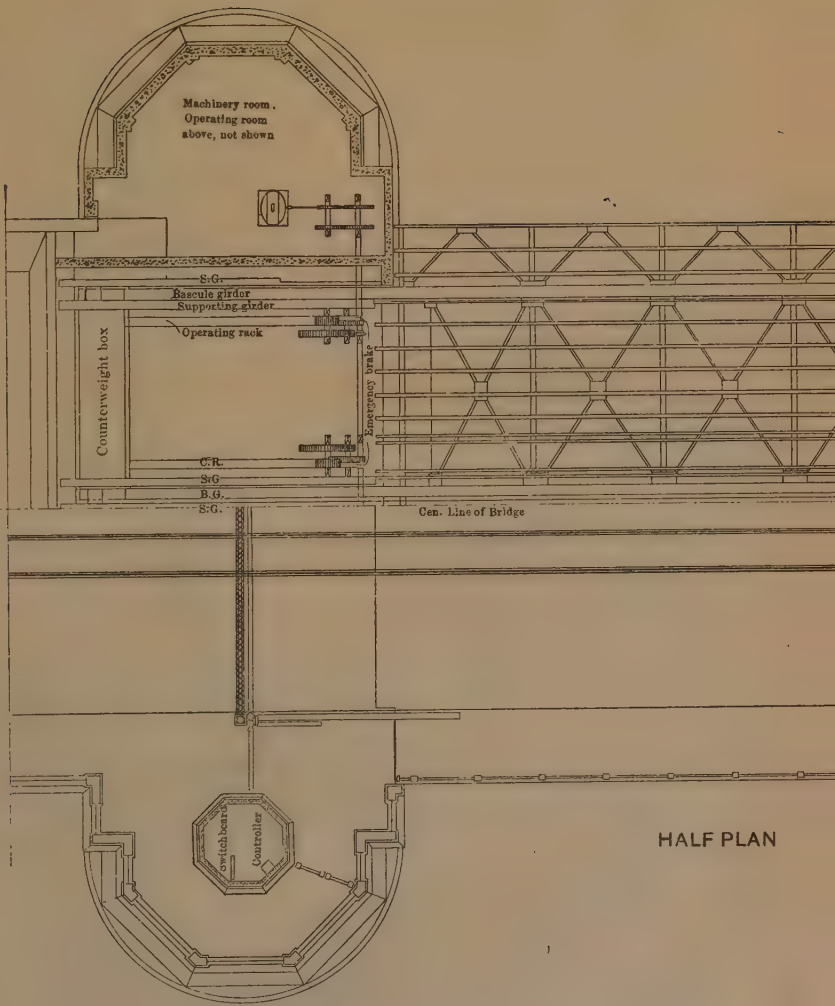
OHIO STREET BASCULE BRIDGE
BUFFALO. N.Y.

FIG. 3.

operating machinery, is the design of T. E. Brown, M. Am. Soc. C. E.

The bridge illustrated in Fig. 4 is a design proposed by the writer for a similar structure, the idea being to secure a tower which would be symmetrical and attractive in appearance, and also to produce an economical design. In this design, the cables are attached to fixed points in the trusses, and pass around sheaves of the proper diameter, being securely fastened thereto. The counterweights are carried by chains, built up of steel plates and pins. These chains pass around, and are wound upon, spiral sheaves mounted upon the same shafts as the first-mentioned sheaves.

The reason for using chains instead of cables for hanging the counterweight is that such chains can be wound around a drum of

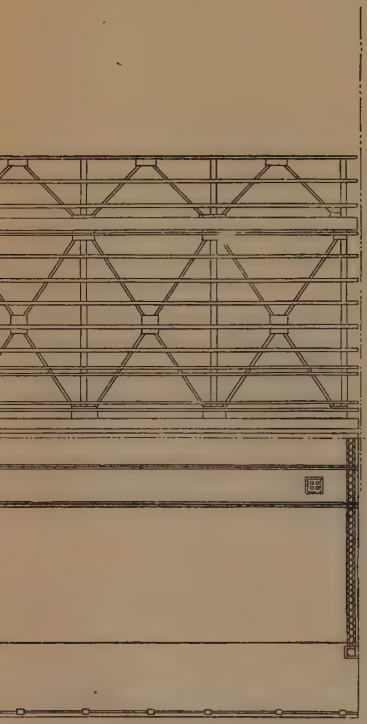

Machinery room.
Operating room
above, not shown
S.G.
Bascule girder
Supporting girder
Operating rack
Counterweight box
Emergency brake
C.R.
S.G.
B.G.
S.G.
Cen. Line of Bridge
Switchboard
Controller

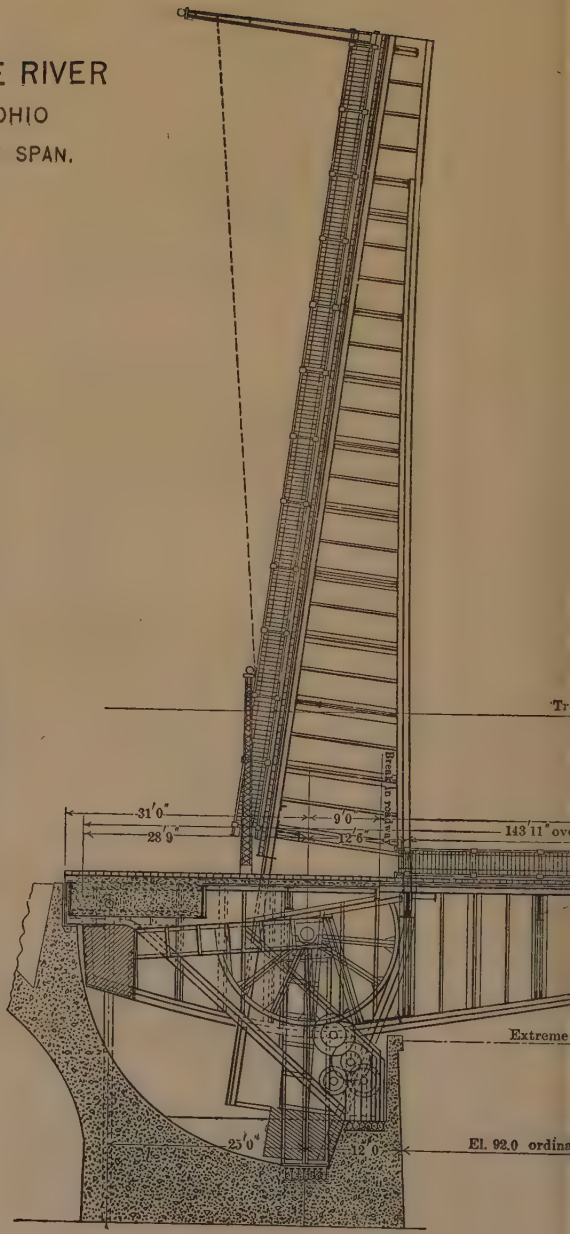
HALF PLAN

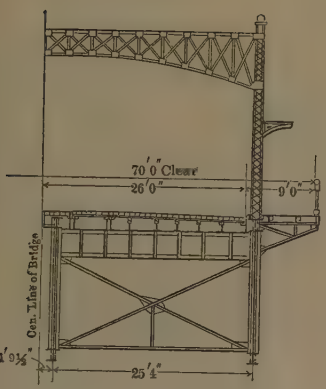
BRIDGE OVER MAUMEE RIVER

ON CHERRY ST., TOLEDO, OHIO

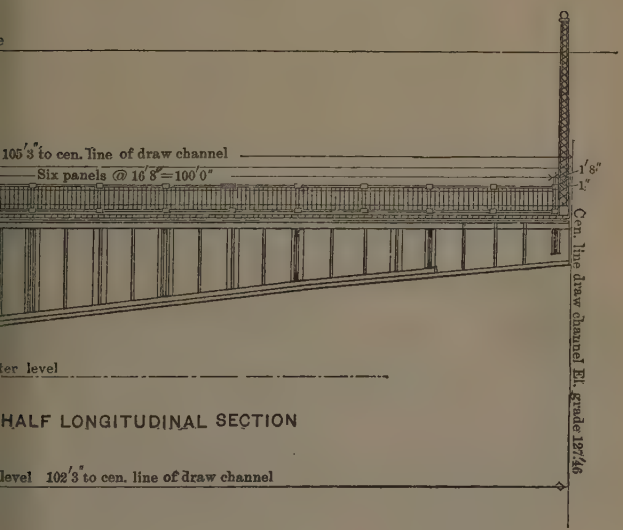
GENERAL DRAWING OF DRAW SPAN.







ALF CROSS-SECTION THROUGH BASCULE GIRDERS

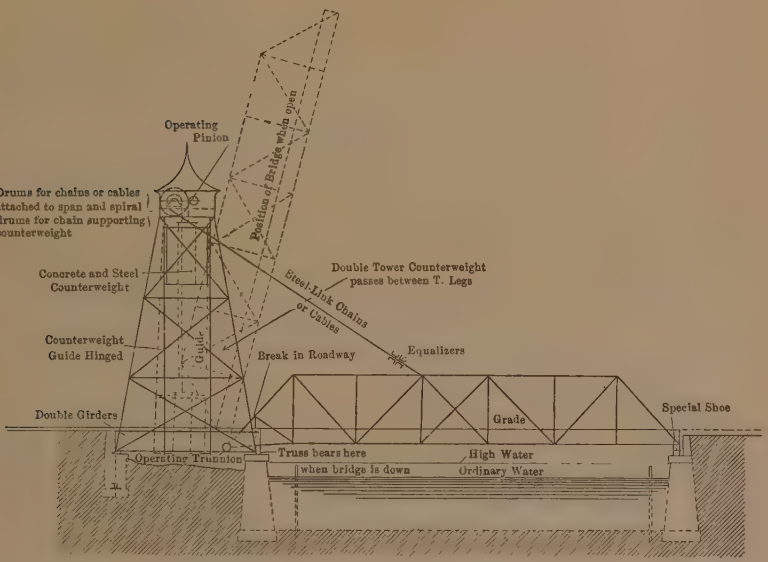


HALF LONGITUDINAL SECTION

level 102′3″ to cen. line of draw channel

very small diameter, much smaller than would be advisable in the case of cables. Mr. Watson.

A further advantage of this design is the simplicity of the operating mechanism, for, by placing the trunnion far enough back, so that the forward, or positive, moment of the span will always be in excess of the backward, or negative, moment of the wind pressure, it is possible to operate the bridge by a pinion engaging a gear attached directly to the shafts which carry the balancing sheaves, and operated by the usual electric motor and gear trains.



THE WATSON BASCULE BRIDGE

FIG. 4.

With such a design, it is practicable to use a counterweight of less than half the weight of that ordinarily used on bascule bridges having attached counterweights, with a consequent reduction in stresses in the structure and in the machinery, and less load on the foundations.

In discussing these designs with other engineers, the writer has discovered that there exists, among bridge engineers, a very strong prejudice against the use of cables and chains, and it is this prejudice to which he desires particularly to call attention, in order to discover, if possible, the reasons therefor. It has been his experience, and is his firm belief, that nothing will give more satis-

Mr. Watson. factory service than chains and cables for such uses, provided that they and their sheaves and fastenings are proportioned properly.

The writer uses a value of 800 lb. per sq. in. for the bearing of axle steel on phosphor-bronze in the case of slow-speed bearings, instead of the 2 000 lb. given by the author.

Plate XXXII illustrates a simple, trunnion, deck-bascule span, designed by the writer for the City of Toledo. It is the writer's opinion that the simple trunnion type for deck bridges has been neglected by engineers, suffering, perhaps, from the fact that it is not a patented type, and, therefore, has not been strenuously advocated by patentees. There are doubtless cases, where, by reason of limited clearance conditions, it is advisable to hinge the counterweight, in order to make its path of travel conform to the said clearance conditions, but, when it is possible to attach the counterweight to the moving span securely without increasing its amount materially, it seems to the writer that the advantage of simplicity of design far outweighs any advantages to be gained by hinging the counterweight.

In regard to controllers for operating double-leaved bascule bridges, the author's specifications call for controllers of the series-parallel type. Now, as it is usually desirable, and required, that a single operator shall be able to operate both leaves from one side of the bridge, controllers of this type require a great number of wires of large capacity to be carried in conduits under the opening. Cannot this result be attained better by using automatic controllers operated by two master switches, which would require much less wiring under the river, and, it seems to the writer, be a safer method of control?

Mr. Hughes. W. M. Hughes, M. Am. Soc. C. E. (by letter).—The writer, in the capacity of Designing and Consulting Engineer, has recently completed, for the Chicago and Alton Railway Company, over the South Fork of the Chicago River, a double-track bridge of the Page bascule type, having a single leaf 150 ft. in length (Plates XXXIII and XXXIV).

With regard to the proper wind pressure to be used in the design of bascule bridges, the writer knows of no reliable information, but this should be available, considering the number of bascule bridges which have been built during the last few years. From observation during the operation of the Chicago and Alton Bridge, with a wind velocity exceeding 30 miles per hour, the ammeter indicated but little increase in the power required. The writer believes 15 lb. per sq. ft., corresponding to a velocity of from 50 to 55 miles per hour, to be ample.

In bascule bridges of the Page type the operating machinery is carried by longitudinal and transverse girders, the latter being connected to the inside counterweight girders, and the power is

PAGE BASCULE BRIDGE OVER SOUTH BRANCH OF CHICAGO RIVER, AT BRIDGEPORT, FOR THE CHICAGO AND ALTON, SANTA FÉ, AND ILLINOIS CENTRAL RAILROADS.

transmitted to the movable leaf by a pinion engaging in a rack which is bolted to the track girder, the latter being connected to the chords of the trusses of the movable leaf. The shaft, to which this pinion is keyed, also carries the rollers, which transmit the counterweight to the track girder. This shaft consists of a hollow steel casting, 24 in. in diameter, the hole being 17 in. The size of this shaft is fixed by the area for the bearing of the rollers, which have phosphor-bronze bushings, the allowed pressure being 1 200 lb. per sq. in. These rollers make less than three revolutions in opening or closing the bridge. Mr. Schneider's specification allows, for moderate speeds, only 600 lb. per sq. in., and, for trunnion bearings, 2 000 lb. per sq. in. The writer is of the opinion that the former is too low and the latter too high. In the design of the Chicago and Alton Bridge, he used 1 200 lb. for the trunnion, more particularly for the purpose of insuring proper lubrication, this being more difficult in the case of the trunnion from the fact that it makes less than one-fourth of a revolution in opening or closing the bridge. Mr. Hughes.

The rule given by Mr. Schneider for the size of keys would hardly be applicable to a 24-in. shaft, which would require two keys, each 2 by 4 in. The keys used for the 24-in. shaft in the Chicago and Alton Bridge were 2 by 2 in., and were considered ample. This, however, may be a special case, as the diameter of the shaft was fixed to give the proper bearing area for the rollers.

When electric motor power is used for the operation of bridges, the writer is of the opinion that it is desirable to use two motors, each of sufficient capacity to handle the bridge, possibly at a somewhat reduced speed, with one motor only in service. This adds slightly to the first cost of the bridge, but practically nothing to the cost of operation, and makes far less the chances of the bridge being thrown out of service, due possibly to the armatures burning out, or some other accident to the motor. When two motors are used, each with sufficient capacity to operate the bridge, it would be hardly reasonable to design the machinery with the usual factor of safety for the full capacity of both motors; although, no doubt, some additional allowances should be made, say 50%, which would provide amply for depreciation due to wear.

J. P. Snow, M. Am. Soc. C. E. (by letter).—This paper is the most practical guide to the design of movable bridges with which the writer is acquainted. The author's conclusions are sound, but it is to be feared that the limitations placed by him upon certain types of swing bridges will be taken, by some, to mean more than was, perhaps, intended. Mr. Snow.

Jack-knife and shear-pole bridges can be readily built with clear openings of 50 ft. This will accommodate an immense amount of navigation under certain regulations. Wider openings are needed on navigable rivers, but around harbors, where draw-bridges are

Mr. Snow. oftentimes very numerous, narrow openings are, on some accounts, distinctly advantageous.

Shear-pole draws cannot be used to advantage except for narrow highways and single-track railways. Two tracks can, of course, be accommodated by two independent structures swinging in opposite ways. Jack-knife draws may be coupled up for four tracks, and, by using two structures swinging in opposite directions, eight adjacent tracks may be handled, as is done on the Boston and Maine Railroad at Boston.

The unbalanced feature in these two types of swing bridges is their worst point. While swinging, they must be held by guys or braces, or both. The lack of a stable attachment for these supports limits the available span of such structures. Proper connection between these attachments and the heel of the trusses is the key to the successful operation of these bridges. Their advantages are: small first cost, quick operation, and less space required outside the tracks than for other kinds of swing bridges.

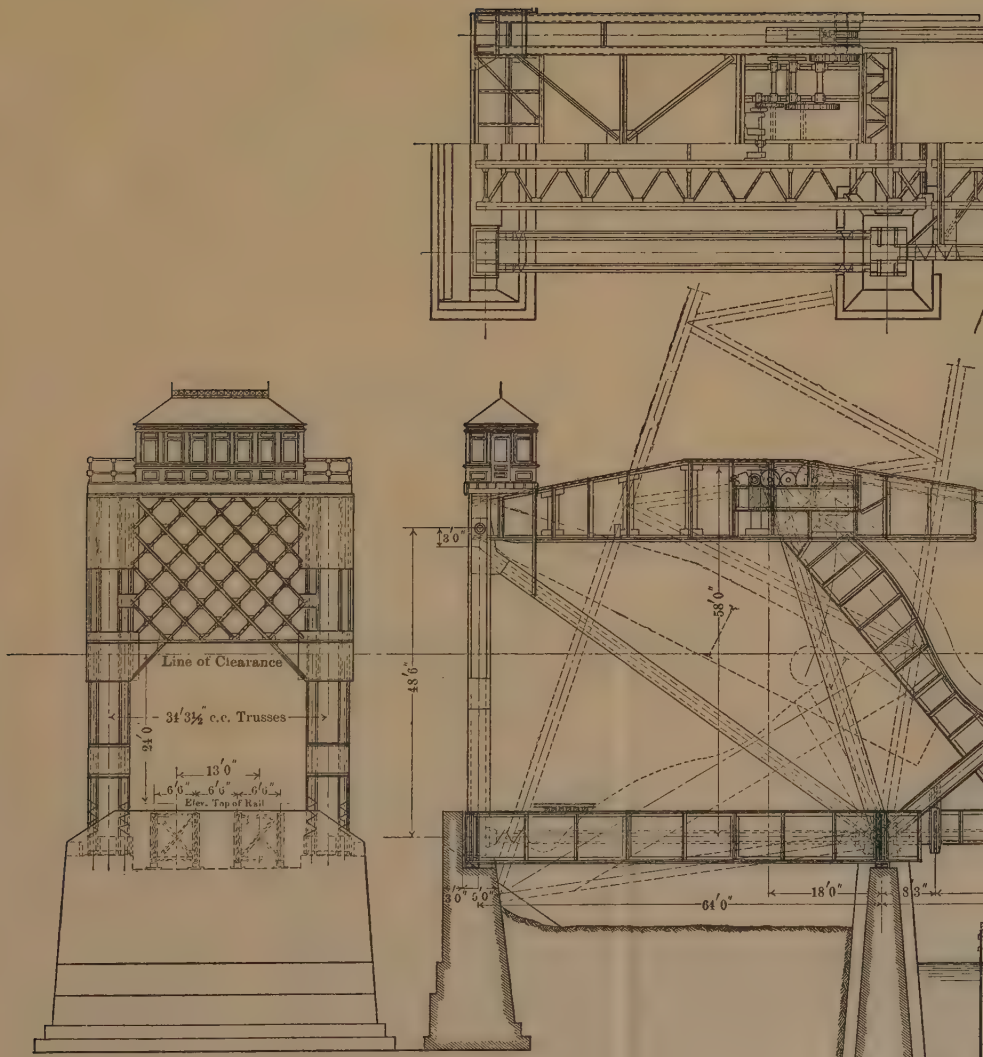
The rim-bearing swing bridge deserves an advocate who will class it a little higher than the author, and, no doubt, in the discussion, an advocate will appear who can handle the subject much better than the writer.

It is true that less power is required to turn a center-bearing bridge than one bearing on a rim; and, if the bridge is to be turned by hand, the former type should be selected. If mechanical power is used, the difference in the amount required is of little importance. Rim-bearing bridges will give trouble if the pier is not firm and intact; and either kind will give trouble if it is not properly built.

A circular drum is no doubt an excellent and very natural feature to carry the upper track of a rim-bearing bridge, but it is not essential. It is not unusual to attach the upper track to the carrying girders and to corner girders attached to them. In this case it is well to make the top track a horizontal plane and set the roller axes inclined. If the diameter of the rollers is made small, excellent results are obtained.

It is essential that the pier for a rim-bearing bridge should be of masonry which will remain in place and hold rigidly the parts attached to it. With this condition fulfilled, very good results have been and can be obtained without connecting the center and lower tracks, as recommended by the author. Such connection, although excellent in itself, will be somewhat difficult and objectionable with stone masonry. With concrete, on the contrary, the bracing can be built into the masonry, and the objection pertaining to stonework will be avoided.

There are in service many rim-bearing bridges which are giving excellent results, and they can be built to do this, if the details are good, without unusual refinements of workmanship.

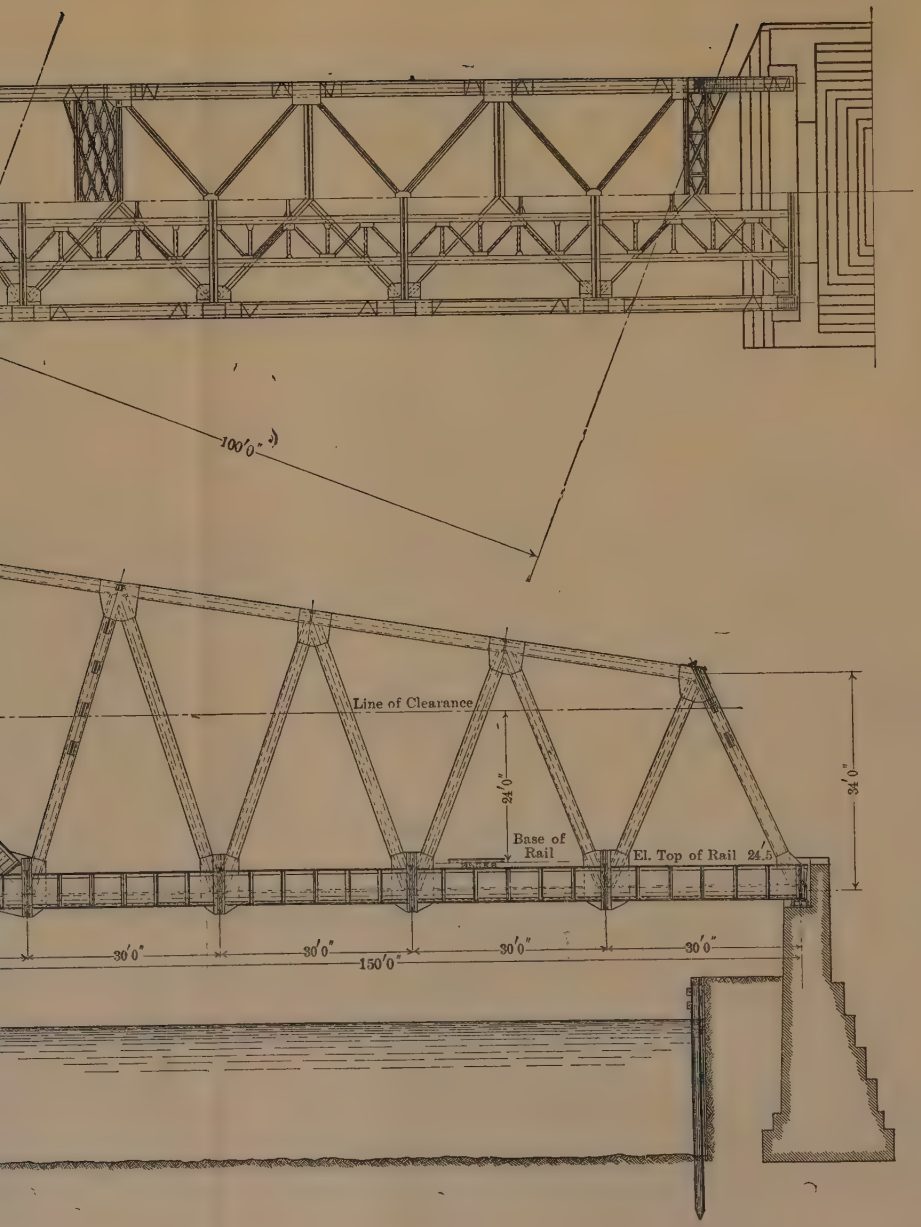


GENERAL PLAN

PAGE BASCULE BRIDGE

OVER SOUTH FORK OF CHICAGO RIVER

C. & A. RY.—C. M. & N. RY. AND A. T. & S. F. RY.


100'0"
Line of Clearance
24'0"
34'0"
Base of Rail
El. Top of Rail 24.5
30'0"
30'0"
150'0"
30'0"
30'0"

THEODORE COOPER, M. AM. SOC. C. E. (by letter).—The character of the foundation and masonry of the pivot pier must often determine the question of preference between a "center pivot" and a "rim-bearing" swing bridge. Any uneven settlement will disable a rim-bearing structure. A pivot pier with a hearting of inferior masonry is unsuitable for a center-bearing bridge. Many of the unsatisfactory results in the past have been due to neglect of these points. Mr. Cooper.

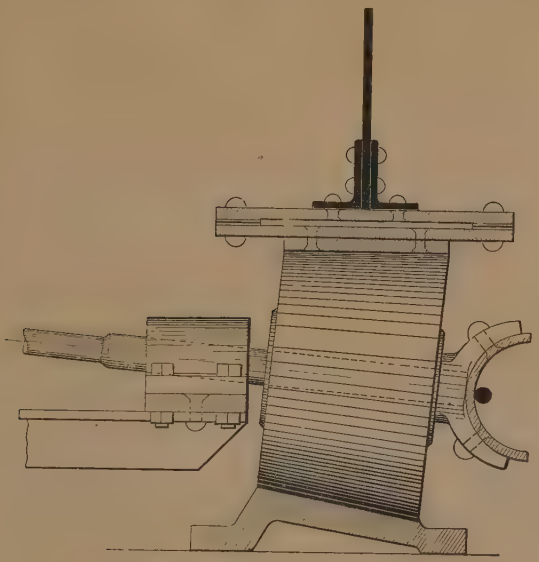


FIG. 5.

Rim-bearing tables, also, as a general rule, have been built upon a false mechanical assumption: that a set of coned wheels will run true between two coned rings, one of which is rigidly held in place by being secured to the masonry, while the other or upper ring is attached to a more or less flexible structure, the bridge floor and trusses. The least distortion of this upper ring makes it impossible for the coned wheels to run true. The result is a distorting and racking strain, upon the radial arms and the guiding center, which is frequently very severe and destructive.

This difficulty is readily overcome, as was done by the writer in 1883 (Second Avenue Bridge over the Harlem River, New York City), by canting the coned wheels, so that the top line is horizontal and all the coning is put in the lower ring segments. (Fig. 5.) Distortion of the bridge cannot affect the perfect action of the wheels, as such distortion can only take place in a horizontal direction.

Mr. Cooper. Fig. 5 also shows the spring plates used to distribute the load more uniformly over the several wheels.

The frictionless wedge, Fig. 6, used first on this bridge, the writer still thinks has some merit. He also thinks toothed gear very unsuitable for movable bridges, and that it should be avoided wherever possible. The back-lash of the teeth allows so much free movement at the ends of the trusses that the bridge is not under full control in high winds. With both rim and center-bearing draw-bridges, he has preferred to parbuckle them with wire ropes operated by hydraulic machinery.

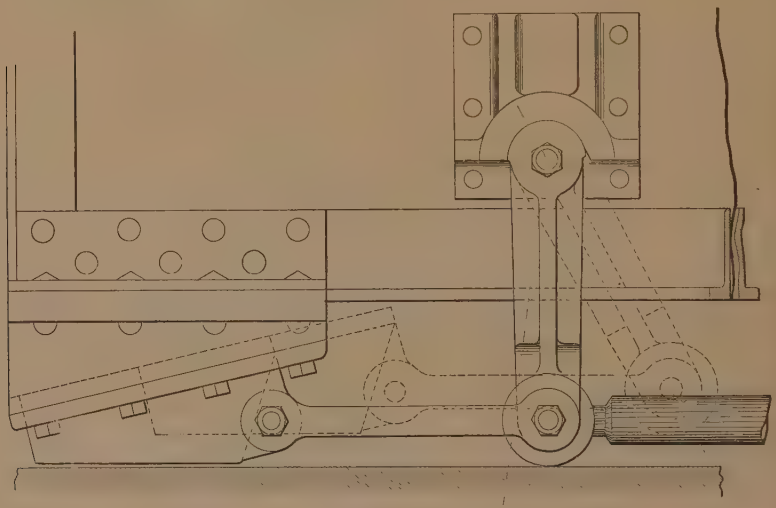


FIG. 6.

In swing bridges it is usually considered desirable to fit them for swinging 180 degrees. While this has the merit of allowing the bridge to open one arm in advance of the vessel, and to follow it and close the opening with the other arm, it is accompanied with more or less danger in the case of railroad bridges. Any shifting of the center or of the tracks, or any unequal expansion of the trusses may interfere with the proper matching of the rails, when the bridge is swung 180 degrees.

Mr. Christie. JAMES CHRISTIE, M. AM. SOC. C. E. (by letter).—Mr. Schneider deserves the thanks of the profession for his painstaking labor in dealing with this subject which involves so much intricate detail.

Bearings.—The specifications imply that bearings of unhardened steel may be used if provided with metaline plugs. When both surfaces are of soft steel, the particles have such a decided tendency to interlock or seize, however efficient the lubrication may be, that it is

safest to prohibit the use of this material in direct contact, excepting occasionally, for brief periods, and at slow velocities. Mr. Christie.

Toothed Gearing.—Spur gearing should be constructed and mounted in such a way that the pressure is distributed over the whole width of the tooth, even with uncut teeth. The old assumption, that the pressure will probably be borne on one corner of a tooth, is unwarranted in good practice.

The author's statement, that "Gear wheels should be designed on the assumption that one tooth transmits the whole pressure," is true for pinions with a minimum number of teeth, say twelve, but, as the number of teeth is increased, some reinforcement is added to the resistance of the teeth. This depends somewhat on the shape of the tooth used, but, approximately, it amounts to a permissible increase of pressure of 3% for each successive tooth, over the minimum number of twelve. While specifying involute teeth, it would be desirable to add the words, "shaped to an accepted interchangeable standard."

The specifications (Table 1) give working unit strains in bending for cast iron of 6 000, 4 000, and 3 000 lb. per sq. in., and 4 000 lb. is adopted as a basis for the strength of toothed gearing. Owing to the great difference between the tensile and compressive strengths of cast iron, its resistance to bending is high, as compared with metals of much higher tensile strength. For example, an old specification for ordinary cast iron—and one readily met- -is a requirement of a center-breaking load of 2 500 lb. on specimens 1 in. square, and 12 in. between supports. This is equivalent to a modulus of rupture of 45 000 lb., therefore, when the bending strain on teeth is limited to 4 000 lb., it is about one-eleventh of the ultimate bending strength of ordinary cast iron.

For several reasons, it is desirable to keep the pitch of spur gearing as small as is safe, and the formula in the specifications involves a greater pitch than is known to be satisfactory in current practice. It is known that some reduction of tooth pressure is requisite as the velocity increases, and this decrease appears to be approximately in the ratio of the square root of the velocity. The following formulas give results of about one-fifth of the ultimate for the respective materials, and have stood satisfactorily the test of prolonged practice:

$$P = \frac{6\,000\,p}{\sqrt{v}}$$ for ordinary cast iron;

$$P = \frac{8\,000\,p}{\sqrt{v}}$$ for strong bronze;

$$P = \frac{10\,000\,p}{\sqrt{v}}$$ for steel castings, 60 000 to 70 000 lb., tensile strain;

$$P = \frac{16\,000\,p}{\sqrt{v}}$$ for steel 100 000 lb., tensile strain.

Mr. Christie.

In these expressions, P = the working pressure on the tooth, in pounds per inch of face; otherwise, the notation is the same as in the specifications. The lowest speed is assumed as 50 ft. per min., and no increase of pressure is allowed for lower velocity. Soft steel, owing to its tendency to seize and cut, should not be used in high-speed gearing. The rules apply to pinions with the minimum number of teeth, and, when this number exceeds twelve, an increase of pressure, as heretofore given, may be allowed.

The author states that the same values shall be permitted for cast iron and bronze, but this would depend much on the kind of bronze. It might be suggested that this equality be limited to common brass, and that a higher value be assigned to high-grade bronze, as suggested in the foregoing rules.

Permissible Unit Pressures for Moving Surfaces, etc.—The limitation of pressure depends on such a variety of conditions that it is impossible to frame general rules on the subject. Calling p the pressure, in pounds per square inch, and v the rubbing velocity, in feet per minute, the product, pv, in practice, is found to vary from less than 50 000 to more than 400 000. It is believed that this product may be greater when p is small and v is large, than in the opposite case, where the velocity is comparatively slow and the pressure is high. Assuming that the surfaces in contact have been fitted so accurately, or gradually worn by judicious service, as to bear uniformly, then the value of pv depends on: The certainty of lubrication, the practicability of artificial cooling, the amount of wear that can be permitted, and the constancy of pressure in one direction, etc.

For example, for crank pins and similar bearings, where the pressure alternates, and ample facility is given for lubricants to penetrate between the rubbing surfaces, a value of 400 000 for pv is common and satisfactory. Whereas, in the neck bearings of rolls in modern rolling mills, with the usual unfavorable conditions attending such service, $pv = 400\,000$ is common practice, but is accompanied by a rapid wear of bearing boxes which could not be tolerated in machinery where greater permanence of the bearings was essential.

As a general rule, for ordinary journal bearings, at moderate velocity, $p = \frac{70\,000}{v + 20}$ corresponds with good practice. The coefficient in the numerator can be increased to 100 000 and more for high velocities without undue wear, or, when the pressure alternates and the velocity is high, as with crank pins, etc., this coefficient can be carried safely between 300 000 and 400 000.

On the contrary, with cross-head slides and similar parts, where, for manifest reasons, wear must be small, the figure should not exceed 50 000.

Bearing Values for Surfaces in Motion.—The values given by Mr. Schneider are generally judicious and safe, excepting the provision for the step-bearings of pivots, viz., $p = \frac{160\,000}{n\,d}$. Even with the best material, or unless a stream of lubricant is forced between the step-plates, this unit pressure appears to be excessive. To avoid overheating, and provide reasonable endurance, one-half the specified pressure would be safer. It might be suggested that the coefficient be reduced to 80 000 for moderate speeds, or between 100 000 and 120 000 for high speeds. Mr. Christie

For center-bearing turn-tables, both conical rollers and step-plates occasionally have proved troublesome. Short cylindrical rollers, placed radially between parallel plates, are now used in machinery, with apparently good results, notwithstanding the seeming objections to this method.

The unit pressures for cylindrical roller bearings, allowed by the specifications, are conservative, the figures for the different materials averaging about one-fourth part of the respective elastic limits, on the lines of greatest pressures; and instances are known where the pressure has been carried nearly up to the elastic limit on roller bearings at low speeds, without trouble.

How much the pressure should be reduced as the velocity increases, on ball and cylindrical roller bearings is unknown, and there is a diversity of opinion on the subject.

Steam Motors.—Why should the piston speed of the engine be limited to 200 ft. per min.? At least double this piston speed accords with safe practice, and the low piston speed specified, implies larger and more wasteful engines than are necessary.

J. W. Schaub, M. Am. Soc. C. E. (by letter).—The author deserves a great deal of credit for presenting this elaborate and valuable paper—the first of its kind ever published in the United States—respecting a subject on which so little has been written. Mr. Schaub

In selecting the type of movable bridge for any existing condition, the swing bridge, by all means, should have first consideration; and if the conditions are such that only one opening is required, so that the pivot pier can be located near the shore, a bridge with unequal arms should be selected, the short arm being counterweighted by some comparatively cheap material, such as concrete. A number of heavy double-track railway swing bridges have recently been built on this plan, which deserves the highest consideration as it presents certain advantages which are not generally understood.

As to the turn-table to adopt for a swing bridge, the center-bearing type should always receive first consideration. The author was probably the first to use this form of turn-table, in the United

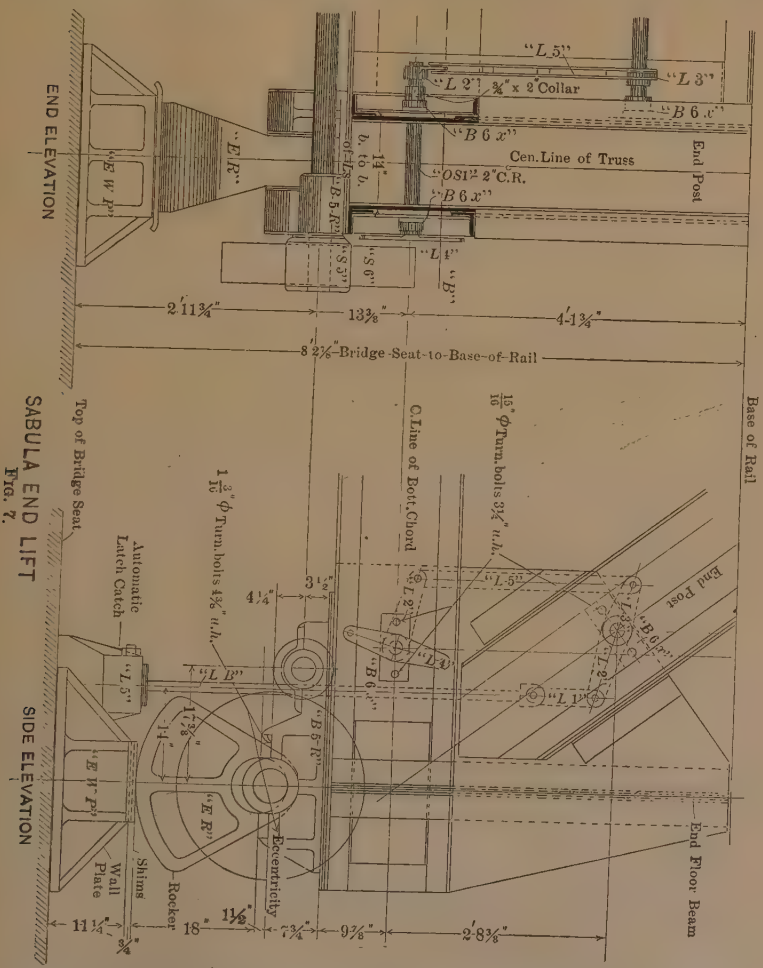
Mr. Schaub. States, as applied to long and heavy spans, so that his preference is well known, but his argument in its favor is not strong enough. When the advantages of the center-bearing swing bridge become fully understood, it will be used to the exclusion of all other types. As the author says, it is only in such cases as for long double-track spans, or spans where the width of roadway is very large, that the center-bearing turn-table becomes impracticable, owing to the great depth required for the cross-girders directly over the pivot.

Another difficulty encountered heretofore has been the suspension of the entire load from the pivot by adjustable hangers, and its distribution equally among them. This difficulty disappears by superimposing the load on the pivot, which should be previously set to the required level, and, by throwing the vertical adjustment into the side bearings, no adjustment is required in the pivot. Also, by making the side bearings adjustable, the wedges are entirely eliminated, thereby simplifying the operating mechanism. In this case four side bearings are used, instead of two, placed equi-distant and forming a square upon the four corners of which the bridge finds a reaction for the moving load only. At the same time, these supports serve to steady the bridge when open or closed, and also shorten the span for the moving load. Moreover, the center pier now becomes square instead of round, thus requiring less masonry and costing less per yard. In addition to this, where a protection is to be provided, up and down stream, the wales can run on the full face of the pier. These advantages are fully realized in a double-track bridge, where the diameter of the center pier becomes an important item in determining the length of the span required to give a certain opening.

In a recent case, requiring a single opening 150 ft. wide, if a rim-bearing turn-table had been used, the diameter of the center pier would have been 45 ft., and the length of the bridge would have been 350 ft. By using a center-bearing turn-table, as above described, the center pier became 36 ft. square, requiring 20% less masonry; and, by using unequal arms, the length of the bridge was reduced from 350 to 250 ft., involving a saving of 10% in the superstructure. Moreover, in this case, the distance from base of rail to masonry was limited, so that a rim-bearing turn-table would have been impracticable.

The trailing wheels, from 18 to 20 in. in diameter, as given by the author, should be of increased diameter, for the longer spans, and the writer would suggest that their diameter, in inches, should equal the length of the span, in feet, divided by ten, but should never be less than 18 in. The larger wheels will reduce the friction, or resistance to turning, and are more in keeping with the large wheels used in center-bearing bridges abroad.

Regarding the end lift, the writer prefers the eccentric rocker, or lift, known in the West as the Sabula end lift (Fig. 7). When the eccentric shaft turns, the ends of the span are raised or lowered by the amount of eccentricity in the shaft. This form of end lift permits the end of the span to move freely, when the length changes, Mr. Schaub.



SABULA END LIFT
FIG. 7.

owing to expansion or contraction, without sliding on the pier. This design originated with the late C. Shaler Smith, M. Am. Soc. C. E., and was first used more than twenty-five years ago in the bridge over the Mississippi River at Sabula, on the Chicago, Milwaukee and St. Paul Railway. This form of end lift has all the

Mr. Schaub. advantages of the wedge, and is particularly applicable to long spans, and all spans that are rarely opened, inasmuch as the horizontal movement at the end of the span is not transmitted to the masonry.

As the author says, in center-bearing bridges with unequal arms, it is an advantage to have the end lift only under one end. This should be confined to the long end; and it can be shown that, if the short end is made less than one-half the length of the long end, the short end will actually rise when the long end is free, so that it is not necessary to give the long end an excess of weight in order to tilt the bridge when swinging.

Rail-lifts should be avoided in all cases, as they are a menace to the safety of a train passing over them, as was shown in the Atlantic City disaster. No track rail on a bridge should be loose, especially at the ends of a swing span. Instead of rail-lifts, the writer uses rail-wedges, which are really movable rail-splices, thereby making the rails continuous, and at the same time serving to line up the track rails. An automatic latch is used for lining up the bridge, but the rail-wedges are used to line up the track rails, absolutely, as the exact alignment of the bridge is not of vital importance. At the same time, these rail-wedges are interlocked with the distant and home signals, so that it is impossible for the operator to break the continuity of the track rails without showing the signals at danger. This is the ideal rail-splice for the ends of a swing span.

Regarding the power required to operate a swing bridge, the formulas and the coefficients for friction given by the author are to be commended, and are practically equivalent to those used by the writer. No mention, however, is made of the collar friction on the hubs of the rollers in a rim-bearing swing span. This resistance to turning represents the effort necessary to keep the rollers in line on the track, and forms an important item in estimating the power required to operate a rim-bearing swing span. This was found to be the case in the experiments made by Messrs. Boller and Schumacher on the Thames River Bridge. For example, assuming the values for collar friction at 0.10 and 0.05 for starting and moving, respectively, and assuming the value for rolling friction at 0.003, the collar friction on a 300-ft. swing span recently built was found to be 8% more than the rolling friction.

The resistance to overcome the inertia of a bridge to turning forms an important item, and varies directly with the time or velocity. The writer made some experiments recently on the Adams Street Bridge, in Chicago, with the view of determining by ammeter readings the percentage of power required to overcome the inertia, as compared with the total power required, and the following are the results:

Mr. Schaub.

The first operation required 90 sec. to turn 90 degrees.

Average of four readings during acceleration.... 40 amperes.
" " " " " uniform motion. 5 "

Power absorbed by inertia = 87.5% of total.

The second operation required 80 sec. to turn 90 degrees.

Average of five readings during acceleration..... 32 amperes.
" " " " " uniform motion. 5 "

Power absorbed by inertia = 84.4% of total.

A 25-h. p. railway-type motor was used; voltage, direct-current, 600 volts; date, June 1st, 1907; temperature, 50°; wind, north, blowing 10 miles per hour at right angles to the bridge.

In a 300-ft. swing span recently built, the writer estimated the power required to overcome the inertia to turning the bridge, through 90° in 90 sec., as 83% of the total effort at the rack circle, assuming the time for acceleration to be 30 sec., so that the proper theoretical consideration, in view of the foregoing experiments, cannot be very far from the truth. It should be explained, that, in view of the large power required to overcome the inertia, no provision need be made to overcome wind pressure; but this does not apply to bascule bridges.

Under details of design for machinery, the author specifies claw couplings for shafting. The writer prefers ordinary flange couplings, as they have no lost motion. In some cases where the torsion in the shaft was very small, he has used a pipe or sleeve bolted through at right angles to the shaft. This serves very well, especially in cases where the room for the shaft did not permit a flange coupling to be used.

Under unit strains, values are given for both torsion and shear. Inasmuch as torsion produces a true shear, no distinction should be made between torsion and shear, so that the values should be given for shear only. The values given for journal bearings are sufficiently low to provide for all speeds of less than 150 rev. per min., so that no provision need be made for heating and seizing in the shafting of a movable bridge, as such speeds are rarely exceeded, except perhaps in the motor shaft, where in any event the maker usually provides a special bearing.

The provisions for the electrical equipment are excellent, and will serve as a guide for any case that may arise.

The specifications or requirements given for special metals, such as phosphor-bronze, steel castings, steel for axles, etc., are excellent, and should be commended. The author deserves the thanks of all who have felt the need of a guide of some kind in designing movable bridges. His paper is of inestimable value.

Mr. Gibbs.

GEORGE GIBBS, M. AM. SOC. C. E. (by letter).—The following comments on Mr. Schneider's very interesting paper are confined entirely to the electrical features.

Electric Motors.—As the paper recommends the method to be used in computing the power of the machinery for operating a bridge, it would seem to leave a blank space in the motor specifications for the rating of the motor. As the duty of the motor is in all cases intermittent, its continuous capacity is not important, and it is sufficient to specify that it shall be capable of developing the maximum torque required during the length of time necessary to open the bridge without heating beyond a specified amount. To provide for contingencies, it is desirable to specify an over-load requirement, but, as in all cases this would be for a very short time, the heating at this over-load is unimportant, but the commutation should not be bad. The specifications could also state the voltage of the circuit used to supply the motors. It is never satisfactory to give merely a load requirement without limiting the temperature, as the term "injurious heating" is too indefinite. It is permissible, however, to use the term "injurious sparking" when referring to commutation, as inspection after operation readily shows whether the sparking has really been injurious to the extent of pitting or burning the copper, insulation, or brushes.

It does not seem to be necessary to require duplicate spare parts for each motor, and the writer suggests changing this requirement, and calling for spare parts for motors of each different size.

Controllers.—The paper specifies series-parallel control with drum controllers. While this is certainly very satisfactory in operation, the writer does not feel that, for all classes of bridges, small as well as large, where it is not necessary to have two economical running speeds, this type of controller should be specified. Although drum controllers are very satisfactory, there are a number of dial-type crane controllers in very general use which are serviceable and satisfactory.

Resistance.—The principal requirement of resistances is that they shall have sufficient capacity to permit of operation of the motors on the resistance without causing damage by over-heating. As some types of resistance will stand without injury much higher temperature than others, it is difficult to give a limiting temperature, as in the case of motors, but as all parts of the resistance usually admit of more complete inspection than a motor, it is not so indefinite if the specifications merely prohibit "injurious heating."

A form of specification for motors and controllers covering these suggestions is submitted herewith.

Regarding cables, wire, grounds, switch-boards, etc., it is more

difficult to give a general specification which will cover all capacities which may be required and all voltages. In regard to the cables, it is hardly necessary to require the supply cable to be separate from the return cable. In fact, the writer believes it to be the more common practice to lay submarine cables as twin conductors. The stranding depends somewhat on the size of the conductor, and, while less than 19 strands (say 7) would be satisfactory for small cables, 19 or even 61 would be desirable for larger ones. The thickness of the insulation depends largely on the voltage. Mr. Gibbs.

Under the heading, "Wiring," the paper first calls for the wiring to be done on porcelain insulators, but later states that all wiring shall be drawn into loricated iron conduits. Wiring on porcelain insulators in places exposed to water or moisture, is the most satisfactory method as regards insulation, but it is not protected mechanically as well as when the wires are run in iron conduits. The latter method of wiring is constantly growing in favor, and even in very damp places, if a good rubber wire is used and the conduit is put up in a water-tight manner, as is easily possible, the result is quite satisfactory. Attention is called to the fact that the word "loricated" is a trade name, and refers to the output of a particular manufacturer. There are several other makes of conduit which are equally good.

Under the heading, "Meters," a 600-volt voltmeter is required and a 300-ampere ammeter. These requirements, of course, are not general, as the voltage of the circuit may be different from this, and the capacity of the motors may require a larger or smaller ammeter.

The switch-board is required to be placed at an angle of 45° with the wall of the operating house. If the house is small, placing the switch-board at this angle may materially decrease the available working space in the house, and it is not objectionable to have the switch-board placed in a vertical position.

In general, specifications for wiring, cables, and appurtenances should state that all work should be done in accordance with the requirements of the National Electric Code for the particular class of work.

Suggested Form of Specifications for Electric Motors for Movable Bridges.

Type.—The motor shall be of the .....-volt railway type, series wound, water-proof, with slotted, drum armature and form-wound armature coils. It shall be of a standard commercial type in common use in order that repair parts may be readily obtained.

Mr. Gibbs.

Capacity.—The motor shall be required to develop ..... h. p. at the armature shaft for a period of ..... min. without temperature rise in any part exceeding 50° cent., as measured in the manner recommended by the Standardization Committee of the American Institute of Electrical Engineers. This is to be known as the full-load capacity. The speed of the armature shaft at this capacity, with ..... volts at the motor leads, shall be not greater than 600 rev. per min. At this output, the operation of the commutator shall be sparkless. The motor will be required to operate at 50% greater load for 10 min. without injurious sparking at the commutator.

Gearing.—The motor frame shall have two bearings for the countershaft, and shall have a forged-steel, cut pinion, keyed to the end of the armature shaft and secured by a lock-nut. One cast-steel, cut gear, bored and key-seated for attachment to the countershaft, shall be furnished with the motor. The gear and pinion shall be covered by a sheet-steel or malleable-iron, split gear-case, supported by the motor frame and completely covering the gear and pinion. An opening, with a hinged cover, shall be provided in the gear-case for inspection and oiling. The gear ratio shall be such that the full-load speed of the countershaft will not be more than 125 rev. per min.

Spare Parts.—For each size of motor furnished, the contractor shall supply the following spare parts: One armature, one field coil, one pinion, one gear, and one set of brushes.

All these parts shall be finished and fitted in such a manner as to admit of being installed in their respective places without further fitting or adjustment.

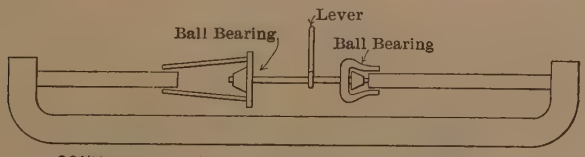
Mounting of Motors.—The motors shall be mounted in such a manner as to admit of easy access for inspection and repairs; they shall be supported securely by brackets or suitable foundations.

Controllers.—The controllers for all motors shall be located in the operating house. The controllers shall be of the reversing type, with magnetic blow-out, and shall be capable of varying and maintaining the speed of the motors throughout the entire range desired, without injurious sparking, and without shock due to sudden variation in speed. All parts shall be of sufficient capacity to operate the motors continuously at full-load torque without a rise in temperature greater than 20° cent., and to operate at 50% overload for 10 min. without a rise of temperature greater than 40° cent.

Resistances.—Resistances shall be preferably of the cast-grid type, and of such capacity that the motor can be operated continuously at any point of the controller when developing full-load torque, or for 10 min. when developing 50% over-load torque, with-

out sufficient rise in temperature of the resistance to cause deterioration of any part. The resistances shall be mounted so as to admit of free ventilation and be without injurious vibration. Mr. Gibbs.

R. Murray, Jun. Am. Soc. C. E. (by letter).—The writer has been especially interested in Mr. Schneider's specifications for ball bearings, inasmuch as he had occasion some time ago to investigate the use of such bearings for swiveling heavy loads. The loads referred to, while in no sense comparable to a draw-bridge reaction, are still by far the largest for ball bearings which have yet come to the writer's attention, ranging up to 70 000 lb. As a result of this investigation, the writer designed a bearing which he had constructed in duplicate to allow for testing. The test of the bearings was then made on the large eye-bar testing machine at the Ambridge plant of the American Bridge Company, with the object, first, of placing such an excess load on the bearings as would give some assurance of freedom from failure under ordinary loads, and second, of ascertaining the approximate amount of friction under this excess load. In this test, the bearings were arranged in the machine substantially as shown on Fig. 8, and a load of 95 000 lb. was applied to each bearing. Under this load, a torque of approximately 4 000 in-lb. was required to turn both bearings. The starting friction for each bearing, therefore, under these conditions, was about 0.008 of the load. Mr. Murray.



CONVENTIONAL SKETCH SHOWING ARRANGEMENT OF BALL BEARINGS IN TESTING MACHINE
Fig. 8.

Since the test above referred to, one of these bearings has been in use for more than a year under working loads ranging up to 50 000 lb., and the writer has been advised that the cups in this bearing have not worn, and that none of the balls has broken. The speed of rotation during testing and in actual use for bearings was very slow, and did not exceed 2 rev. per min. Fig. 9 shows the bearing in detail.

From consideration of the foregoing facts, the writer believes that the formula presented by Mr. Schneider will give results on the safe side—certainly for balls up to $1\frac{1}{8}$ in. in diameter. It should be noted that the friction value given above was secured when the load greatly exceeded the permissible load given by Mr. Schneider's formula.

Mr. Murray.

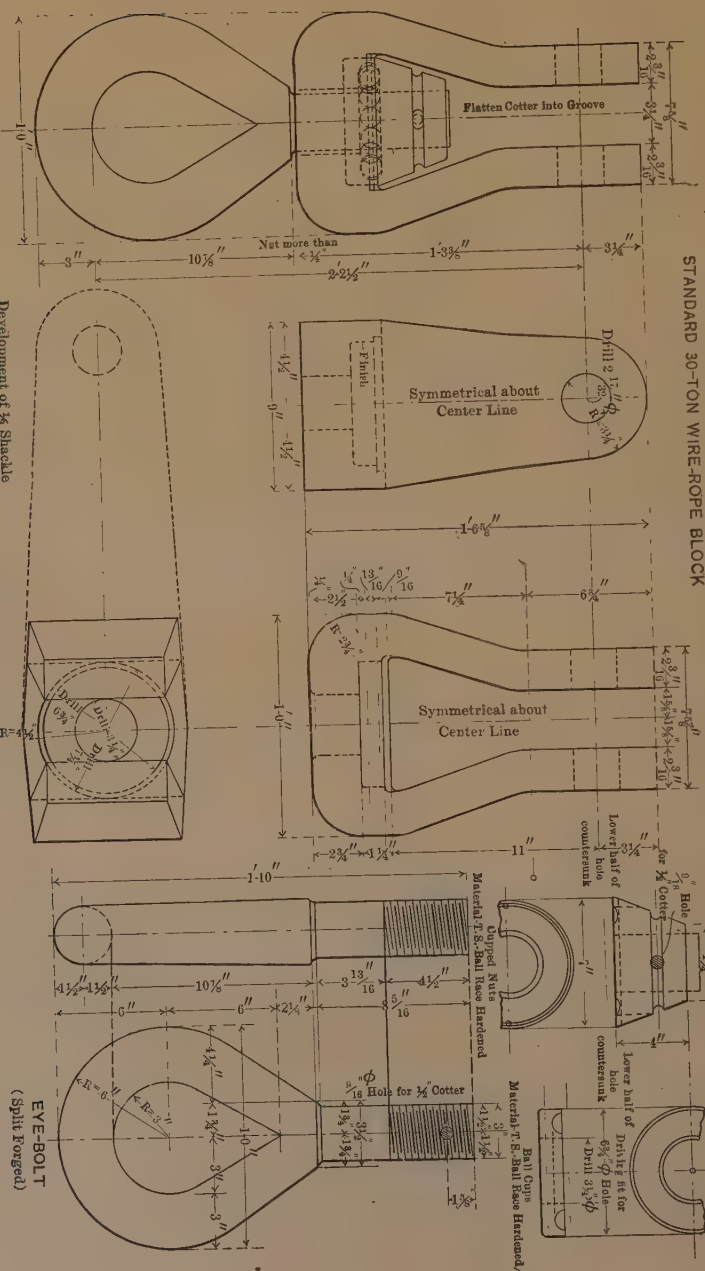


FIG. 9.

C. C. Schneider, Past-President, Am. Soc. C. E. (by letter).—In reviewing the discussions, the writer wishes to express his gratitude for the interest taken in the subject of his paper and for the many valuable suggestions offered and for errors corrected. Mr. Schneider

Mr. Worcester's description of the jack-knife draw is complete and comprehensive, and covers many points which were omitted by the writer in describing this kind of bridge.

Mr. Dart's suggestions in reference to pavements, and his practical experience in the maintenance of bascule bridges, form a valuable contribution to the literature on movable bridges.

Rim- Versus Center-Bearing Swing Bridges.—As regards these two principal types of swing bridges, the consensus of opinion appears to be in favor of the center-bearing type. The writer is aware, as Mr. Snow remarks, that there are many rim-bearing bridges in existence which are giving excellent results, while, on the other hand, there are some center-bearing bridges which are giving bad results. However, if both types are equally well designed, the center-bearing bridge is preferable as offering the following advantages: Economy of first cost, economy of maintenance, and greater stability.

The writer's experience is also supported by the fact that whenever a railroad has used the modern center-bearing swing bridge once, it has continued to use this type, and no other, up to the present day. The Pennsylvania Railroad alone has built nineteen bridges of this type since 1890.

The writer agrees with Mr. Cooper that the rim-bearing type, as a general rule, has been built upon false mechanical assumptions; and that if the top track of a rim-bearing swing bridge were made in a horizontal plane, and the roller axis set inclined, as designed by Mr. Cooper for the Second Avenue Bridge, New York City, and also suggested by Mr. Snow, it would do away with the tendency to work the center pivot loose, and it would not be necessary to connect the center with the lower track.

Mr. Cooper's arrangement of spring plates, used to distribute the load more uniformly over several wheels, is also worthy of consideration.

The writer agrees with Mr. Gay that the more rigid and unyielding the base, as well as the moving parts of a turn-table, the easier and cheaper will be its maintenance; and that the proper preparation of the top of the pivot pier under the track is of the utmost importance for a rim-bearing bridge.

The writer has always used wedges to support the ends of the girders in the center of center-bearing swing bridges; but the arrangement, suggested by Mr. Schaub, of using side bearings with vertical adjustment in place of wedges, thereby entirely eliminating the wedges and simplifying the operating machinery, appears to have some merit, and is worthy of consideration.

Mr. Schneider.

Bascule Bridges.—The designs of bascule bridges submitted by Messrs. Watson and Hughes illustrate some good construction in movable bridges of this type, and the writer wishes that more designs had been submitted, showing some of the other types which have proved equally satisfactory.

The writer can see no objection to the use of cables and chains in connection with bascule or any other kind of lift bridge.

As regards the proposed wind pressure to be used in the design of bascule bridges, the usual practice has been to assume 15 lb. per sq. ft., as mentioned by Mr. Hughes, which is probably sufficient, but the writer adopted 20 lb. per sq. ft., as he considered it rather an advantage to provide for an excess of power and strength of machinery.

Selection of Design.—The writer considers the two conditions which Mr. Smith suggests adding to the three already mentioned, as precluding the use of swing bridges, of sufficient importance to warrant their addition.

Power Required to Operate Movable Bridges.—The tests of the power required to operate swing bridges, submitted by Messrs. Greiner and Schaub, seem to indicate that the actual power required to overcome the frictional resistance and the acceleration is well within the limits of that given by the writer's formula.

The simple rule given by Mr. Smith for mechanically-operated bridges, "to allow 1 h.p. for each 15 tons of weight to be swung," has been verified by the writer, and found to provide ample margin for practically all contingencies, and is probably as good a rule as the more complicated formulas, as an excess of power is an advantage.

Mr. Smith's suggestion in reference to calculating the strength of gearing for bridges operated by hand-power, by assuming the power of one man at 125 lb. with as many men as it is possible to place at the hand-lever, would certainly give the gearing strength enough to make it safe against all accidental possibilities.

End-Lifts.—The writer, in later years, has given the preference to wedges for the end-lift of swing bridges; formerly, he also used rollers and toggle joints. He adopted wedges because they offer a better and more substantial support at the ends, and also on account of the preference given to them by those who had charge of the maintenance of the bridges. However, he has no prejudice against other kinds of end-lifts which he knows have proved satisfactory, such as the toggle-joint arrangement used on the Vancouver-Portland Bridges, and the roller arrangement used on the Rock Island Bridge designed by Mr. Modjeski, also the Sabula end-lift submitted by Mr. Schaub, consisting of eccentric rockers.

The frictionless wedges used by Mr. Cooper on the Second Avenue Bridge, New York City, combine the advantages of the wedge and roller arrangements, and have proved satisfactory in practice.

Rail-Lifts.—Mr. Modjeski and Mr. Schaub object to the provision in the specifications calling for rail-lifts. The writer acknowledges that the rail-lift is not an ideal arrangement, and may be entirely superseded by safer devices in the future. Mr. Schneider.

The rail-lock used by Mr. Modjeski on the Vancouver-Portland Bridges, and the one described by Mr. Schaub, are improvements on the rail-lift.

Rack and Track.—Mr. Worcester evidently has understood the writer's recommendation, that the rack and track segments should be cast in one piece, as covering all cases, but this is specified only for those of center-bearing bridges where the track is very light. This has been the writer's practice for many years, and was taught him by experience. In cases where the track is very light, it is preferable to cast the rack and track in one piece in order to get better castings, as the track segments, if connected with the rack, will keep the rack from warping, not only in the pattern, but also during the process of casting; and, in the case of center-bearing bridges, the combination rack and track is just as easily replaced as a rack segment alone (if not more easily), as the trailing wheels do not interfere with this operation.

The writer believes that, instead of specifying that the patterns for special gearing, such as rack and pinion, should be furnished with the bridge so that broken parts can be easily replaced, it would be better to require that some additional parts be furnished with the bridge, as manufacturers generally prefer to keep the patterns, which may be used again for other bridges.

Couplings.—Mr. Gay and Mr. Schaub object to claw couplings for connecting shafting for machinery, on account of lost motion. In specifying claw couplings, the writer did not mean loose couplings such as are sometimes furnished by bridge shops; and his specifications require that all workmanship shall be in accordance with the best machine-shop practice. A properly fitting claw coupling has no appreciable lost motion. There may be no apparent trouble with rigid couplings, but, as the bridge deflects and the shafts have to revolve while they are forming a curve, they cannot revolve without twisting, bending, and wearing out their bearings. This has been confirmed by the writer's observations. He prefers to have less play in the bearings, and, at the same time, have the shafts perform their operations freely.

In his early practice the writer used rigid couplings because this was the usual method in connecting line shafts in shops where, after the bearings are properly lined up, they will remain in the same position.

Shrouded Pinions.—Mr. Dart objects to shrouded pinions, preferring to increase the width of the face of the tooth for the purpose of increasing the strength of the pinion. The writer can see no advantage

Mr. Schneider. in making the face more than twice the pitch of the teeth. As the teeth in cast gearing are likely to bear only on one corner, their strength is calculated for that condition, and, therefore, the increased width of the tooth would not add materially to the strength if it were more than one and one-half times the pitch. The only other way to increase the strength of the pinion teeth is, as Mr. Dart suggests, to increase the thickness of the teeth of pinions engaged in racks at the expense of the rack teeth.

Keys and Set-Screws.—Mr. Gay asks what virtue there is in placing keys and set-screws at an angle of 120° with each other. The same question may be asked in reference to placing the same at 90 degrees. This matter is of small importance, as long as they are not opposite each other. The writer specified 120° simply because it appears to be the modern machine-shop practice, and the reason for changing this from the former practice is to have the hub bear on the shaft at three equidistant points.

Unit Strains.—Mr. Schaub is of the opinion that, inasmuch as torsion produces a true shear, no distinction should be made between torsion and shear; and that the values should be given for shear only. The writer agrees with Mr. Schaub, and, as the differences between the values of shear and torsion as given in the table are so small that this change would not materially affect the strength of the shafting, the values for torsion should be omitted and the fourth item in the table should be called "shear and torsion."

Bearings.—The specifications provide that bearings of steel on steel shall not be used for any rotating or sliding surfaces, unless provided with metalline plugs. Mr. Christie thinks that it is safest to prohibit the use of soft steel in direct contact with the same metal in moving surfaces. The writer thinks this point is well taken, and, therefore, the clause referring to metalline plugs might be omitted.

Mr. Smith asks the question: "Has anyone ever had any trouble with cast iron sliding on cast steel, when used for wedge surfaces?" The writer has in some instances used cast-iron wedges sliding on steel bearings which have proved satisfactory.

Strength of Toothed Gearings.—Mr. Christie is of the opinion that spur gearing should be constructed and mounted in such a way that the pressure is distributed over the whole width of the tooth, even in uncut teeth. The fact remains, however, that the uncut tooth does not bear over the whole width, unless made to do so by long-continued wear. The writer, therefore, believes it to be safe practice to compute the strength of the tooth for a bearing on one corner only.

The formula given in the specifications for the strength of gear wheels is intended for the minimum number of teeth in the pinion, and, as Mr. Christie, as well as Mr. Hess, suggests, the pressure may be increased with the additional number of teeth. Mr. Christie sug-

gests a permissible increase of pressure of 3% for each additional tooth over the minimum number of 12. This would make an increase of 100% for 33 additional teeth, which the writer considers excessive, but recommends the adoption of the more conservative Lewis' formula for increasing the value of y if the number of teeth exceeds the minimum of 12. The value of the coefficient, y, as modified to agree with that specified for $n = 12$, would be $y = 0.1 - \frac{0.6}{n}$, where n = number of teeth in the pinion, which for $n = \infty$ would make $y = 0.1$. Mr. Schneider.

The writer is aware that the formula in the specifications gives a larger pitch than that provided for in the usual machine-shop practice, and that it is desirable to keep the pitch of spur gearing as small as it is safe. This is good practice for gearing in machine tools and ordinary mill machinery, where the motion is regular and more or less continuous, and where a break may be of small consequence. The gearing in movable bridges, however, is subject to irregular and alternate motions, accompanied by shocks and vibrations, and, therefore, it is of the utmost importance to have the machinery absolutely safe, for the reason that a broken wheel cannot be as readily replaced in a bridge as in a factory, but may cause suspension of traffic, which on a railroad is a serious matter.

Mr. Christie's formula for the strength of tooth gearing does not contain the face, or width of the tooth. The writer, therefore, assumes that this formula was intended for a face of twice the pitch. From this it appears that Mr. Christie allows a higher strain on cast iron and practically the same strain on steel as the writer. As the face of the teeth in motor gearing is sometimes four or five times the pitch, the writer thinks this factor ought to be taken into consideration for cut gearing. For ordinary bronze the same value has been assumed as for cast iron, but, as suggested by Mr. Christie, a higher value may be allowed for high-grade bronze. The writer agrees with Mr. Christie that soft steel should not be used in high-speed gearing. He has specified for steel castings, therefore, a minimum ultimate strength of 70 000 lb. per sq. in.

Permissible Unit Pressure for Moving Surfaces.—The permissible pressures given in the table are for ordinary moderate speeds, and a formula is given for reducing these for high speeds. As the terms "moderate speeds" and "high speeds" are very indefinite, the formula is intended to determine up to what speeds the values are applicable, and what they should be if that speed is exceeded. It is found, for instance, that the bearing value given for hardened steel on bronze is 1 500 lb. per sq. in. The formula for higher speeds gives the permissible pressure, $p = \frac{300\,000}{n\,d}$; the bearing value of 1 500 lb. per sq. in., therefore, is limited for journals to $n\,d = 200$, which represents about

Mr. Schneider. 630 in., or approximately 52 ft., per min. If a shaft 10 in. in diameter made 300 rev. per min., or $n\ d = 300$, the pressure would be limited to 1 000 lb. per sq. in. These values agree very closely with those suggested by Mr. Christie.

Messrs. Watson and Hughes are of the opinion that the permissible pressure allowed for trunnion bearings of bascule bridges is too high. Mr. Watson allows 800 lb., and Mr. Hughes, 1 200 lb. per sq. in. Mr. Hughes also thinks that the permissible pressure of 600 lb. per sq. in. for structural steel on bronze is too low. The permissible pressure of 2 000 lb. per sq. in. is intended for journals of axle steel on phosphor-bronze of the special kind mentioned in the specifications for bearings with slow and intermittent motion; while the pressure of 600 lb. per sq. in. is intended for structural steel on ordinary bronze for moderate speeds. A pressure of 2 000 lb. per sq. in. when used in connection with this special kind of phosphor-bronze for journals making only 1 or 2 rev. per min. has given no trouble. Of course, in cases of such high pressures, special arrangements should be made to insure proper lubrication. It might perhaps be well to change the permissible bearing pressure from 2 000 lb., as given by the writer, to 1 500 lb. per sq. in.

Steam Motors.—Mr. Christie asks: "Why should the piston speed of the engine be limited to 200 ft. per min.?" The reason for limiting the piston speed for steam engines, as well as internal-combustion and electric motors, is to obtain more substantial machinery. The high-speed motors are generally too light and flimsy for movable bridges. They get out of order frequently, and the small additional first cost is of little consequence compared with the benefit derived from having motors which can stand the rough usage and the wear and tear to which they are subject in movable bridges.

The opinion of Messrs. Smith and Hughes that two motors should be used in all bridges operated by electricity is shared by the writer, and a recommendation to that effect should be embodied in the specifications.

The writer gratefully acknowledges the valuable contribution by Mr. Gibbs, who has enlightened him on matters referring to the electrical equipment of movable bridges in which he was lacking in experience. The specifications for electric motors, as suggested by Mr. Gibbs, should be substituted for those contained in the writer's specifications.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1072.

ROTATING SCREEN OF POWER CANAL, SALT RIVER PROJECT.

By F. Teichman, M. Am. Soc. C. E.

One feature of the Salt River Project, Arizona, United States Reclamation Service, is a power canal, 19 miles long, having a capacity of 250 cu. ft. per sec., and 240 ft. head. The power developed will eventually be used mainly in pumping for irrigation in the Phœnix Valley. At present the power is used for the construction of the reservoir dam across the Salt River Cañon, which includes the operation of a cement mill. The power is developed in turbine wheels with guide-buckets adjusted by the governor.

The water of the canal carries much sediment, and also a great deal of grass, sticks, etc., especially at times of heavy rains. These floating bodies are likely to lodge in the narrow end of the guide-buckets, and it may occur (as it has occurred) that, if these bodies are not kept out, the governor, in attempting to close the buckets at moments of small demand of power, will break the bucket which is blocked, and such breakage may be followed (and was followed) by breakage in the runner of the turbine.

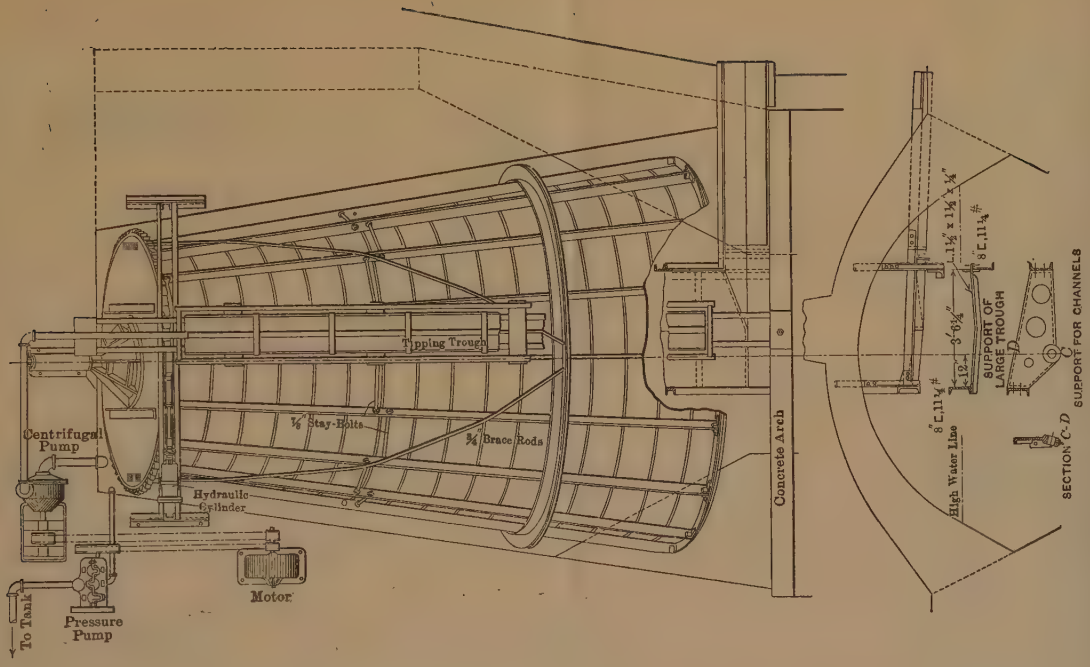
To avoid such accidents it has become necessary to send the water of the power canal through a screen, naturally located at the penstock. This screen is supposedly of a novel design, and, having proven successful in operation, it is thought that a short description may be of interest.

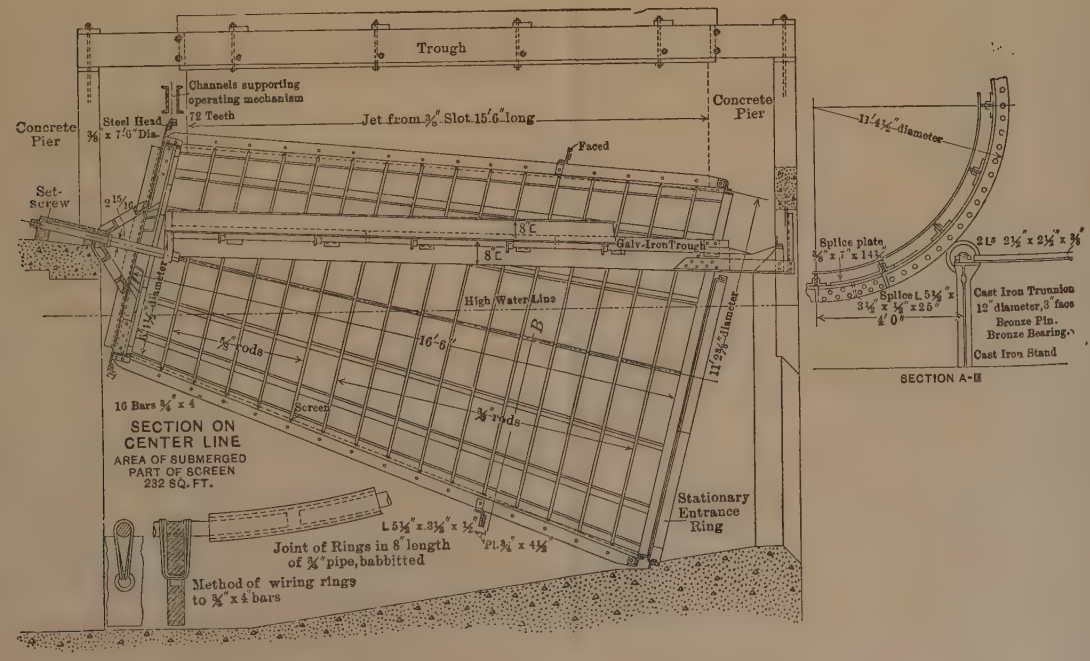
The screen has the form of a truncated cone, the water entering the cone at the base. The axis of the cone is at an inclination of 1 : 4.

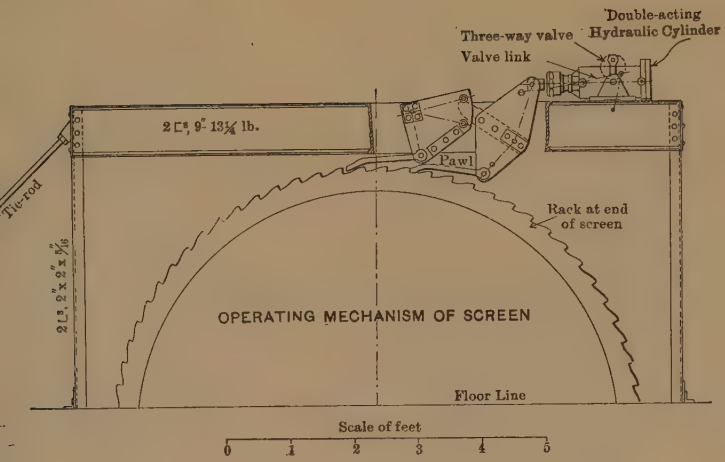
The cone is made to rotate slowly around its axis. To the entrance ring of the cone is fitted (with a small clearance of only ¼ in.) the stationary entrance ring. The screen of the cone is made of No. 18 galvanized wire, ½-in. mesh, and is supported by circular rods (⅝ and ¾ in. in diameter) which in their turn are supported by sixteen ¾ by 4-in. bars, riveted to the steel head of the drum at one end, and to the entrance ring at the other. At a point about two-sevenths of their length from the entrance ring the sixteen bars are connected and supported by the trunnion ring. Between the trunnion ring and the steel head these bars are connected by ⅞-in. stay-bolts. To the steel head is riveted a center casting, bored for a stationary shaft, 3 in. in diameter, held in a cast-iron shaft support. The structure of the screen, therefore, is supported by this 3-in. shaft and by two trunnions in the plane of the trunnion ring. To the steel head are bolted the segments of a cast-iron ratchet ring of 72 teeth; and two pawls, actuated by a hydraulic cylinder, propel the screen, which makes a complete turn in about 1 hour.

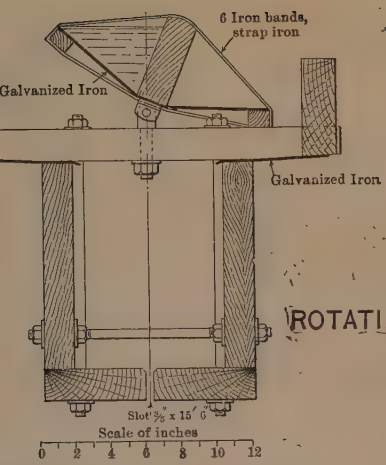
The extension of the 3-in. shaft in the interior of the drum, and a concrete arch over the canal about 16 in. from the drum, support, in the interior of the drum, two 8-in. channels, slightly inclined, on which rests a galvanized-iron trough, 3½ ft. wide and 16½ ft. long, that terminates in a side trough. The motor which turns the screen also pumps water into a horizontal trough above the screen and centrally above the interior 3½-ft. trough. At intervals this upper trough (the tipping trough) discharges automatically, and the water it contained strikes the screen in a jet (⅝ in. by 15 ft. 6 in.), carrying with it into the interior trough any material which had lodged on the inner surface of the screen. Material which does not adhere to the screen is lifted by blades attached to the interior of the screen, and dropped into the interior trough. The wash-water carries the screenings through the side trough over the edge of the canal.

The area of the submerged screen is 232 sq. ft. with the canal discharging 250 cu. ft. per sec. The screen would safely stand a difference in level of the water, inside and outside, of 4 ft., which may possibly occur during the first heavy rains of the season.


Tipping Trough
Centrifugal Pump
⅛" Stay-Bolts
¾" Brace Rods
Hydraulic Cylinder
Concrete Arch
Motor
To Tank
Pressure Pump
SUPPORT OF LARGE TROUGH
SUPPORT FOR CHANNELS
SECTION C-D
High Water Line


Trough
Channels supporting operating mechanism
Concrete Pier
Steel Head
72 Teeth
Jet from ¾" Slot 15'6" long
Faced
Set-screw
Galv. Iron Trough
High Water Line
¾" rods
16 Bars ¾" x 4
Screen
SECTION ON CENTER LINE
AREA OF SUBMERGED PART OF SCREEN 232 SQ. FT.
Stationary Entrance Ring
Joint of Rings in 8" length of ¾" pipe, babbitted
Method of wiring rings to ⅜" x ¼ bars
11'4½" diameter
Splice plate
Cast Iron Trunnion 12" diameter, 3" face
Bronze Pin.
Bronze Bearing.
Cast Iron Stand
SECTION A-B





CROSS-SECTION OF TIPPING TROUGH

SALT RIVER PROJECT.

ROTATING SCREEN AT PENSTOCK OF POWER CANAL.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1073.

NOTE ON THE IMPROVEMENT OF THE MISSISSIPPI RIVER.

By W. G. Price, M. Am. Soc. C. E.

With Discussion by Messrs. L. E. Lion, and W. G. Price.

During the past twenty-seven years much work has been done on the Mississippi River between Cairo and New Orleans, but comparatively little has been accomplished in the way of permanent improvement, except in the construction of levees.

The revetting of caving banks by a continuous mattress has certainly not proven to be a permanent form of improvement in many places, and whenever such a mattress is located for a long time in a place where it is opposed to the force of the river flow, it must fail. Much of the mattress which has been placed was constructed in such a manner that, if not carried away bodily by the first flood, it would disintegrate and float away stick by stick, owing to the fastenings being of metal, and soon destroyed by corrosion.

To design structures which will not be carried away has been a difficult problem, but such have been built, and, in the light of experience, still better designs can now be made.

Some years ago the writer was engaged in improving the Harbor of New Orleans. This was done by works which formed what were intended to be fixed points in the bank of the river and could not be

carried away. The writer believed then that this was the proper way to secure a permanent improvement of the whole river. Some of the work at these fixed points was put in with such details of construction that they should have failed before this time; the writer, however, believes that permanent works on this plan can now be constructed which will confine the river so as to prevent it from wandering away from the place where it is desired to hold it, and also greatly improve the navigable depth. These fixed points would have to be located at various distances apart, depending on the degree of curvature of the river, but the number required would not entail a prohibitive expense, provided each one was put in so as to be a permanent work and not require constant repairs, and it is with the view of giving a hint of how to make such work permanent that this paper has been written.

The Mississippi has shown that it has power to destroy and carry away almost everything that Man has constructed to oppose it. By using a little strategy in design, however, the river can be compelled to use its mighty power to dig foundations for permanent improvement works. This can be done, and has been done, by making the form of construction of these works such that the river cannot disintegrate them, or move them bodily down stream. It is known that it can, and will, undermine them, and it is by taking advantage of this undermining power that the point can be made a fixture.

By designing the structure so that only wood and stone, properly bound together, form its permanent parts, the forces acting against it can do nothing but undermine it and sink it in the sand, while more of the same material is piled on top of that which is sinking, and a depth of foundation is soon reached which the river will not cut under. In this way the flowing water will sink a large structure to a depth which will give a permanent foundation, in the same way that a pile is sunk with a jet of water.

The work must be well connected to the levee, and must extend well out on the bed of the river.

When the writer was building the dam from Turnbulls Island across Red River, in Louisiana, he had no fear of the river being able to destroy the work, or cut around it, and he believes now that it was a great mistake to destroy this dam, by dredging a channel through it, after it was more than half completed.

The writer believes that the power of the flowing water in any silt and debris-bearing stream can be utilized and directed, by a properly designed structure, so that it will dig a permanent foundation for such a structure, and that a sufficient number of improvement works have been built, and have been tested for a sufficient length of time, in the Mississippi and other rivers, to indicate the proper details of construction.

DISCUSSION.

Mr Lion. L. E. Lion, M. Am. Soc. C. E. (by letter).—Several years ago, the writer, while engaged on work under the Mississippi River Commission on the Lower Mississippi, had experience with two distinct types of bank protection. These were spur-dikes, placed at selected intervals, and continuous revetments. Both these works were applied only to the concave or bend side of the stream, as no erosion takes place on the points or convex sides of the river. The currents are swift in the bends, and the banks of the river are often composed of materials easily disintegrated by the rapidly moving water, causing recession of the bank line, and loss of land and the improvements thereon.

Spur-dikes—dikes of timber cribs ballasted with stone—are placed normal to the bank of the river, and project into the stream. They are designed to hold intact selected points in the bend of a river, and to direct the current so as to prevent it from eroding the bank.

Continuous revetment protects the river banks from erosion by placing over them a covering of willow mattresses ballasted with stone, after the manner of a concrete or masonry lining in an open channel. The mattresses are built of willow poles and saplings which grow on the banks of the river, and are the most economical form of construction yet devised for continuous revetment.

The method of building spur-dikes and holding fixed points has met with success only where the current is of moderate velocity. These dikes give rise to swift eddies, whirls, and vertical currents, which cause rapid caving, and form deep indentations in the shore line between dikes. If the space between dikes is not protected, this erosion continues until finally the current cuts behind the dikes. To the writer's knowledge, examples of this action occurred in Giles' Bend, Mississippi, and in Carrollton Bend, at New Orleans Harbor, and at both places the continuous revetment has almost entirely supplanted the spur-dikes.

Continuous revetment is more costly than spur-dikes, but it is also more efficacious when properly built and placed. The mattresses should be carried out into the bed of the stream until the section of the river begins to rise to the opposite shore, in order to avoid undercutting at the foot of the revetment. On account of the expense of this work and the limited funds usually available, revetment is often carried too short a distance into the stream, with the result that undercutting occurs at the foot of the mattress, thus causing it to rest on a slope of constantly increasing steepness. The erosion continues until finally a part of the stone ballast holding the mat in position rolls off. The lightened part of the mattress is then free to rise and fall, and bend and twist, with the action of the current until it breaks and floats

away piece by piece. It is difficult to arrest destructive action of this kind, when once begun, and it is often necessary to sink new mattresses over the broken ones. The engineer has no way of knowing that the mattress has failed until it is shown by the caving of the bank. It is advisable, therefore, to take no chances, but to carry the revetment well into the stream. Mr. Lion.

The usual method of procedure is to begin protection near the upper end of a bend and work down stream. It has been found that if a bend is only partially revetted, undermining will take place at the down-stream end of the protection.

The art of protecting the banks of large rivers flowing through alluvial deposits is in a crude state, and, as now carried on, is very expensive, its cost being prohibitive in ordinary cases. Revetment on the Mississippi River at present is justified only for the protection of valuable city fronts and improvements, for the preservation of river harbors, and for the maintenance of important levee lines which, if breached, could only be replaced by costly new lines. It is also sometimes justified in order to avert a threatened cut-off across a narrow point where such a cut-off would disturb the regimen of the river for miles above and below the threatened point, and thus intensify caving.

Iron and steel fastenings, after a time, are entirely destroyed by the action of the Mississippi River water, but the loss of these connections is anticipated by driving a treenail beside each spike in the frames of the mattresses, and by the use of composition wire, which is not affected by the water.

W. G. Price, M. Am. Soc. C. E. (by letter).—When the spur-dikes for New Orleans Harbor were designed, it was expected that the river would erode the banks between them and thus form a scalloped shore, and that wharves constructed in these scallops would slide into the river on account of the weight of the silt which would collect under them. It was understood that a system of spur-dikes, 1 000 ft. or more apart, was not the best system for a harbor which must eventually have a continuous line of wharves, but the appropriations were comparatively small, for the length of river bank to be protected, and it was neccssary at that time to stop whole city squares from melting into the river. Mr. Price.

A continuous mattress revetment, with the mattress 2 or 3 ft. thick, cannot be a permanent bank protection in New Orleans Harbor. The mattress must necessarily be laid on banks which are steep, to the point of caving or sliding down, and, when wharves are built, the accumulation of silt, due to the obstruction of the current by the piling, will cause the bank to slide and wreck the mattress. For this reason, the steep slope of the bank must be reduced, and this can best be done with cribs laid on top of the mattress, such as were originally used in the spur-dikes in New Orleans Harbor for this purpose. This reduces

Mr. Price. the form of construction to a system of spur-dikes so close together that there are no gaps between them.

Composition wire should not be used for a permanent fastening in mattress and crib construction. The writer, by careful tests in the Mississippi River, found that the impact of the sand in the flowing water would cut away all exposed parts of metal fastenings. These tests led to the use of the treenail in spur-dike construction.

The mattress and cribs must be constructed with pockets to hold the rock, so that the rock will not roll out of the pockets until the mattress assumes a vertical position. Such a mattress need not be carried as far out on the river bed as one from which the rock ballast will easily roll off.

When the spur-dikes are located some distance apart, eddies will form between them which will cut into the bank. The force of these eddies grows less the farther they eat into the bank, owing to the increasing distance the water has to travel, thus reducing the slope and therefore the velocity. The extent of the cutting of a bank by such eddies, therefore, is limited.

Spur-dikes, in order to form permanent fixed points, must be built of a size in proportion to the power of the river. The mattress for the Lower Mississippi should probably be not less than 300 ft. wide, parallel with the bank line. In front of towns where wharves are required, the mattress would have to be continuous, and the cribs a very short distance apart. In other localities, with a proper selection for the fixed points, the spur-dikes might average some miles apart. The dikes would cause eddies and much caving of the banks until the river had changed to suit the new limiting conditions. The stronger the dike the farther apart they could be, and, as a result, the more would the river change its bank line before a comparatively fixed course could be established.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

Paper No. 1074.

MUNICIPAL REFUSE DISPOSAL: AN INVESTIGATION.*

By J. T. Fetherston, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. W. M. Venable, Albert A. Cary, E. H. Foster, B. F. Welton, C. Herschel Koyl, Louis L. Tribus, H. Norman Leask, Rutger B. Green, George N. Cole, Edwin A. Fisher, Frederick L. Stearns, William F. Morse, E. B. B. Newton, Rudolph Hering, and J. T. Fetherston.

For a number of years the disposal of municipal refuse in the Borough of Richmond, New York City, has been the subject of much concern to responsible authorities. The first attempt to deal systematically with solid organic waste was made in 1895, when the former Village of New Brighton erected a Brownlee crematory, and collected garbage separated from other refuse. In 1898, the general Department of Street Cleaning introduced the separation system throughout the Borough of Richmond, and ashes, rubbish, and street sweepings were used for filling sunken lots, while garbage within economical hauling distance of a Dixon crematory (erected in 1899) was destroyed, the remainder being mixed with ashes, etc., and dumped on isolated properties. In 1902, when the city charter was amended, the collection and disposal of refuse, with other street-cleaning work, came under the control of the Borough President.

* Presented at the meeting of December 18th, 1907.

The writer was placed in charge of the Bureau of Street Cleaning in 1904, and found the conditions regarding refuse disposal as follows:

1.—The separation of garbage from ashes and rubbish was imperfect. Short-haul ash dumps were sources of complaint. Land for the disposal of refuse was becoming scarce. The cremation of garbage as practiced was costly and unsatisfactory. Little trouble was experienced in the use of street sweepings for filling sunken lots, as macadam roads adjoined paved streets and the sweepings contained relatively small percentages of organic matter.

2.—The loss of short-haul dumping grounds had progressed to the point where provision for power transportation to waste lands, or some other means of final disposition of household refuse, became necessary. Efforts to obtain a reasonable rate of transportation by street railway to outlying properties failed. Examination, tests, and studies of existing garbage crematories offered no hopes of satisfying permanently the requirements of economical and sanitary final disposition. The location, population, and topography of the built-up districts under consideration rendered any means of disposal other than incineration inadvisable.

After a study of available publications on the subject, it became evident that the British method of destroying mixed refuse would solve the problem if the local refuse contained enough combustible material to consume all noxious wastes without nuisance, and provided that a reasonable degree of economy in operation could be attained.

From various books, reports, etc., the following general abstract of English practice in destroying mixed refuse was compiled:

A.—Refuse destructors are centrally located in populous districts, and are reported to be operated without nuisance.

B.—Average British household refuse consists of one-third combustible, one-third water and one-third mineral matter, by weight (Watson). This material is burned in brick furnaces by the aid of forced draft without the use of additional fuel. High temperatures are attained, and the waste gases are brought into contact with steam boilers, thus producing power which is utilized for various purposes. Each pound of average unscreened refuse will evaporate its own weight of water.

C.—The residue consists of clinker, fine ash, and dust, amounting to about one-third, by weight, of the original material. Clinker is

FIG. 1.—COVERED REFUSE COLLECTION CARTS AT WORTHING.

FIG. 2.—TIPPING REFUSE INTO STORAGE HOPPER AT WATFORD.

FIG. 3.—SALTLEY DESTRUCTOR, BIRMINGHAM.

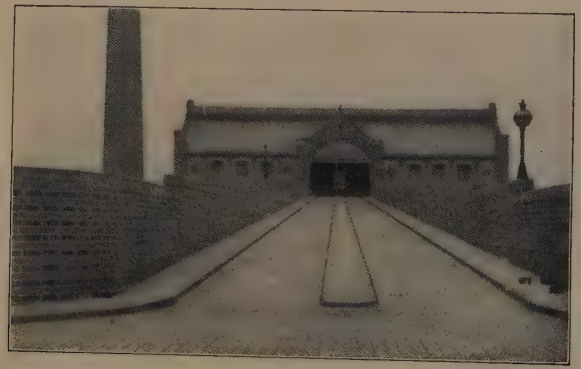

FIG. 4.—LOOKING UP RUNWAY AT SALTLEY

TABLE 1.—Composition of Refuse from Different Cities, Classified Approximately.

Description.	Fine ash, dust and dirt. Percentage.	Clinker. Percentage.	Glass, metal, etc. Percentage.	Coal, coke, breeze, and cinder. Percentage.	Garbage. Percentage.	Rubbish. Percentage.	Paper, straw and fibrous material, vegetables, bones, and offal. Percentage.	Authority.
Average Ash-bin Refuse, England	32.00	.....	3.00	51.00	.....	.....	14.00	Hutton.
London Refuse	52.60	.....	3.575	28.95	.....	.....	14.875	Codrington.
London Ash-bin Refuse	19.51	.....	2.98	64.53	.....	.....	12 98	Russell.
Torquay, Eng., Dec. and Jan., Refuse	25.42	.....	3.17	6.51	52.42	12.29	.....	Goodrich { "Disposal of Towns Refuse" p. 139.
Torquay, Eng., June Refuse	48.05	.....	5.50	28.92	13.32	4.21	.....	" " p. 141.
Berlin, Germany:								
April Refuse	57.12	1.58	7.02	1.33	29.53	3.42	.....	Bohm and Grohn (Hering).
August Refuse	43.19	1.18	9.28	1.53	36.62	8.20	.....	" "
New York	52.30	.....	0.46	28.17	12.20	6.87	.....	Craven (Hering).

Chemical Analysis of Berlin Refuse, by Bohm and Grohn (Hering).

Description of material.	Hygroscopic water. Percentage.	Combined water and carbonic acid. Percentage.	Combustible organic matter. Percentage.	Incombustible. Percentage.
Sifted	10.91	2.54	13.27	73.28
Coarse	26.55	9.53	10.20	53.72
Unsifted (computed.)	17.62	5.54	11.94	54.90

Chemical Analysis of Refuse from Kings Norton (near Birmingham), Eng.

	Percentage.
Carbon	36.80
Hydrogen	0.29
Nitrogen	0.29
Sulphur	0.19
Oxygen	7.30
Ash	41.70
Moisture	12.12
Total	98.69

Theoretical calorific value of this refuse given as 4 500 B. t. u. (From Specification, p. 26.)

Calorific Values of English Refuse Components with Average Percentage of Moisture (Dawson).

Coal	9 334 B. t. u.
Coke	8 000 "
Bones and Offal	5 334 "
Breeze and Cinder	4 000 "
Rags	3 334 "
Paper, straw, fibrous material, and vegetable refuse	2 534 "

useful as an aggregate in making concrete slabs, bricks, etc. If not utilized, the residue makes acceptable filling.

D.—The cost of labor and supervision per ton of refuse destroyed amounts to about 30 cents in England. The economy of the process depends to a great extent upon the utilization of the steam generated.

Up to this time (1904), all available information concerning the composition of refuse from different localities was summarized as shown in Table 1. These figures were so meager and unsatisfactory for the case in hand that only two courses were open: namely, build a destructor, with the risk of failure; or make a thorough examination of the whole question, to determine the best means of disposal should mixed refuse destruction prove unsatisfactory.

The object of this paper is to present the results of an investigation covering:

First, the quantity, composition, seasonal variations, and calorific power of local household refuse, with a series of practical tests in burning mixed wastes; and second, some data, observations, and deductions concerning a number of mixed-refuse destructors visited by the writer.

Part I.

Richmond Borough (Staten Island) has an estimated population of about 80 000, and an area of 57.2 sq. miles. It is divided into three refuse collection districts, two of which, containing the bulk of the population, are served by municipal carts, while the third, comprising the country or sparsely-settled sections, is served by hired carts.

One of the refuse collection divisions, known as the West New Brighton District, was selected as a representative portion of the borough from which to obtain the desired information concerning the problem of disposal. By actual count, there were 4 321 houses in this district, containing about 25 900 people, 90% of whom contribute wastes for removal by city carts.

The plan adopted contemplated obtaining data concerning:

I.—The quantity of refuse contributed per 1 000 inhabitants per day, by volume and by weight;

II.—The seasonal variations in quantity of refuse, by volume and by weight;

III.—The components of refuse, and their seasonal variations;

FIG. 1.—MIXED-REFUSE DUMP AT FOX HILLS, BOROUGH OF RICHMOND, N. Y., JUNE 27TH, 1906.

FIG. 2.—AN OLD REFUSE HEAP AT WORTHING, ENGLAND, PHOTOGRAPHED JUNE 15TH, 1906.

FIG. 3.—REFUSE ON STORAGE, LLANDUDNO, WALES, MAY 19TH, 1906.

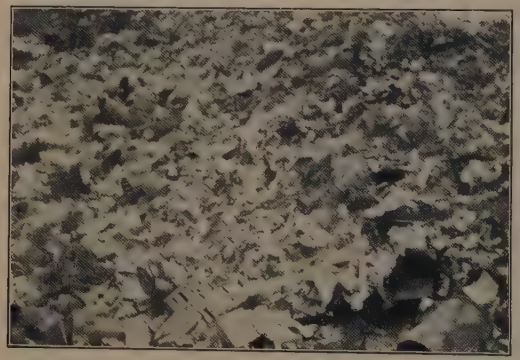

FIG. 4.—NEAR VIEW OF REFUSE AT FOX HILLS, BOROUGH OF RICHMOND, N. Y., JUNE 27TH, 1906.

IV.—The calorific power of refuse, and its seasonal variations; also the seasonal variations in the calorific power of the components of refuse;

V.—The practical incineration of mixed refuse;

VI.—The probable temperature of the gases resulting from the destruction of refuse, and the boiler-power obtainable with seasonal variations.

Definitions.—Household refuse means practically all materials collected by municipal carts from private residences, stores, and small factories. Night-soil, dead animals, and trade refuse, such as building refuse and large manufacturing wastes, are not included.

Ashes consist mainly of the residue from domestic fires, from schools, churches, etc., and include other inorganic material, such as glass, crockery, metallic substances, dust, bricks, stones, earthenware, etc.

Garbage is composed of organic materials—vegetable and animal matter—with water and grease.

Rubbish consists of light combustible articles, such as paper, rags, excelsior, straw, wood, leather, rubber, etc.

I.—Quantity of Refuse.—Table 2 records in convenient form the quantities of different classes of household refuse as collected in the West New Brighton District.

The volumes of the different classes of refuse were obtained by careful records of cart loads. The adopted unit weights were the average results of numerous determinations. Table 2 shows that the average quantity of household refuse collected amounted to about 3.7 cu. yd. or 1.6 tons per 1 000 inhabitants per day during the period considered.

II.—Seasonal Variations in Quantity of Refuse.—Referring to Table 2, it will be noted that the volume of the total collection for any month differs slightly (from 8% above to 12% below the average), while the weight varies from 23% above the average in winter and spring to 30% below the average in summer or fall. The two classes of refuse have opposite variations by weight, "ashes and rubbish" being high in winter and low in summer, while "garbage" is low in winter and high in summer. Unseasonable weather and abnormal variations in the fuel and food supply will cause further differences in the amounts of "ashes," "rubbish," and "garbage."

TABLE 2.—HOUSEHOLD REFUSE AS COLLECTED, WEST NEW BRIGHTON DISTRICT.

Month.	Ashes and Rubbish.						Garbage.						Total Collection.			
	Volume.			Weight.			Volume.			Weight.			Volume.		Weight.	
	Cubic yards.	Percentage of 12 months' collection.	Percentage of total monthly collection.	Tons.	Percentage of 12 months' collection.	Percentage of total monthly collection.	Cubic yards.	Percentage of 12 months' collection.	Percentage of total monthly collection.	Tons.	Percentage of 12 months' collection.	Percentage of total monthly collection.	Cubic yards.	Percentage of 12 months' collection.	Tons.	Percentage of 12 months' collection.
1906.																
January	2 014	9.3	83.1	962.9	10.6	83.5	409	5.7	16.9	190.6	5.7	16.5	2 423	8.4	1 153.5	9.3
February	1 852	8.6	87.5	871.5	9.6	87.6	265	3.7	12.5	123.5	3.7	12.4	2 117	7.4	995.0	8.0
March	2 199	10.2	85.5	1 072.7	11.8	86.0	373	5.2	14.5	173.8	5.2	14.0	2 572	9.0	1 246.5	10.1
April	1 972	9.1	78.3	973.8	10.7	79.3	545	7.7	21.7	254.0	7.7	20.7	2 517	8.8	1 227.8	9.9
May	1 857	8.6	76.4	890.4	10.9	78.7	574	8.1	23.6	267.5	8.1	21.3	2 431	8.5	1 257.9	10.2
June	1 725	8.0	73.7	734.0	8.1	71.9	617	8.7	26.3	287.5	8.7	28.1	2 342	8.2	1 021.5	8.2
July	1 516	7.0	69.9	426.0	4.7	58.3	653	9.2	30.1	304.3	9.2	41.7	2 169	7 5	730.3	5.9
August	1 661	7.7	68.8	417.4	4.6	54.3	753	10.6	31.2	350.9	10.6	45.7	2 414	8.4	768.3	6.2
September	1 677	7.7	65.3	429.1	4.7	50.9	889	12.5	34.7	414.3	12.5	49.1	2 566	8.9	843.4	6.8
1905.																
October	1 567	7.2	66.1	564.7	6.2	60.1	803	11.3	33.9	374.2	11.3	39.9	2 370	8.2	938.2	7.6
November	1 678	7.8	72.0	759.8	8.4	71.4	652	9.2	28.0	303.8	9.2	28.6	2 330	8.1	1 063.6	8.6
December	1 894	8.8	76.7	877.5	9.7	76.6	574	8.1	23.3	267.5	8.1	23.4	2 468	8.6	1 145.0	9.2
Totals	21 612	100		9 079.8	100		7 107	100		3 311.9	100		28 719	100	12 391.7	100
Percentage of total collection		75.2			73.3			24.8			26.7		100		100	
Average amount per 1 000 inhabitants per day		2.77 cu. yd.			1.164 tons.			0.91 cu. yd.			0.425 tons.		3.68 cu. yd.		1.589 tons.	

NOTE.—One ton = 2 000 lb. { Daily collection of refuse, except Sundays and holidays. Average weights per cubic yard: ashes and rubbish = 0.42 ton; garbage = 0.466 ton

III.—Components of Refuse and Their Seasonal Variations.—Mechanical analyses, by weight, of the two classes of refuse shown in Table 2 were made during 1904, 1905, and 1906, as often as other work permitted. Cart loads of "ashes and rubbish," amounting to

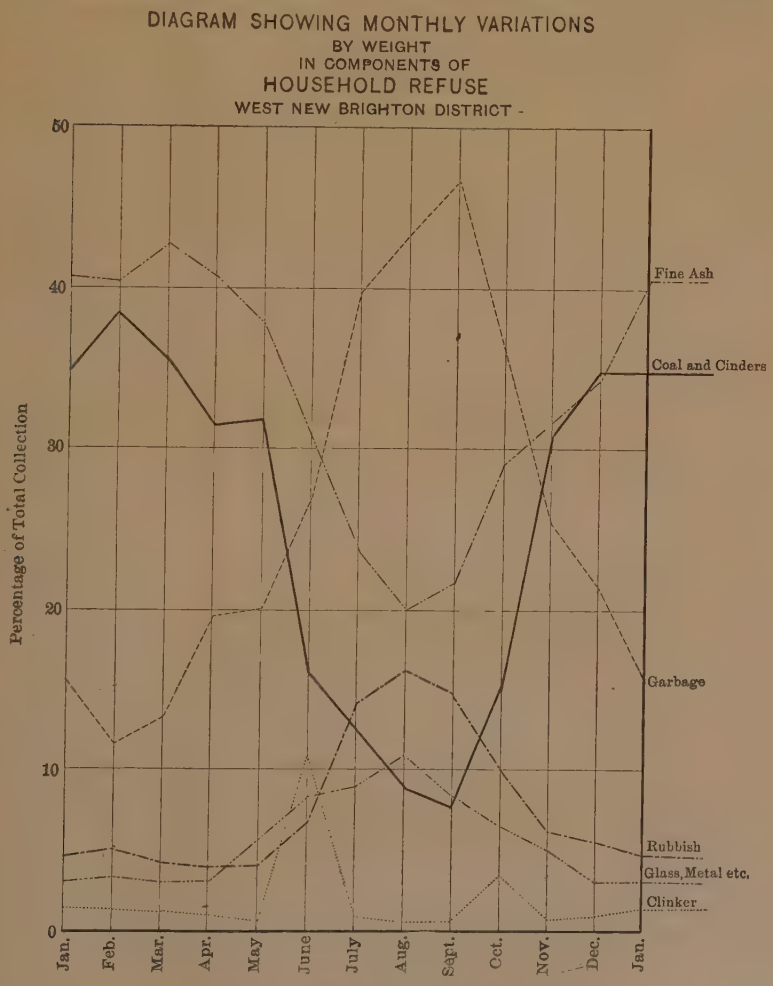

FIG. 1.

107 907 lb., from different portions of the district, were analyzed, and 562 samples of garbage, amounting to 28 101 lb., were separated into components. A $\frac{3}{8}$-in. mesh screen was used to divide the "ashes and rubbish," the material passing through the screen being termed

"fine ash," while "coal and cinders" were the materials rejected by the screen and not otherwise classified as "rubbish," "clinker," "glass, metal, etc.," separated by hand-picking. Small samples (averaging 50 lb. each) of "garbage" were separated into components by hand. The average results of the monthly determinations were used in connection with the figures in Table 2 to compute Table 3 which gives the results of the mechanical analyses. The graphic diagram, Fig. 1, brings out clearly the relative variations in the components of refuse from this district, and, as "garbage" is the primary cause of nuisance, it indicates that September will be the critical month when destroying mixed household wastes. February represents the opposite extreme of low "garbage" and high "coal and cinders." The large percentage of "fine ash" in winter suggests the advisability of removing this material, though the decision in this respect will depend upon practical experience in dealing with the refuse. As "garbage" carries nearly all the moisture in mixed refuse, a series of forty-one evaporative tests was made in order to determine its water content. Samples averaging 39 lb. were dried in a special water-jacketed evaporation pan, with the results indicated in Table 4.

TABLE 3.—Composition of Household Refuse by Weight.

Month.	From Table 2.		From Mechanical Analysis.							
	Ashes and rubbish. Percentage.	Garbage. Percentage.	Fine ash. Percentage.	Clinker. Percentage.	Glass, metal, etc Percentage.	Coal and cinders. Percentage.	Garbage.			Rubbish. Percentage.
							Vegetable. Percentage.	Animal. Percentage.	Free water. Percentage.	
1906.										
January	83.5	16.5	40.5	1.4	3.1	34.7	14.3	0.6	0.7	4.7
February	87.6	12.4	40.3	1.3	3.4	38.3	10.9	0.4	0.3	5.1
March	86.0	14.0	42.6	1.2	3.1	35.5	12.2	0.5	0.6	4.3
April	79.3	20.7	40.8	1.0	3.2	31.5	17.9	0.8	0.8	4.0
May	78.7	21.3	37.7	0.6	5.7	31.8	18.7	0.7	0.7	4.1
June	71.9	28.1	30.7	11.1	8.4	16.2	24.4	1.0	1.4	6.8
July	58.3	41.7	23.8	0.8	9.0	12.6	36.3	1.6	1.7	14.2
August	54.3	45.7	20.0	0.5	10.9	9.0	39.7	1.7	2.0	16.2
September	50.5	49.1	21.7	0.6	8.5	7.7	42.5	1.9	2.2	14.9
1905.										
October	60.1	39.9	29.0	3.5	6.6	15.2	30.9	3.1	1.5	10.2
November	71.4	28.6	31.8	0.7	5.2	30.8	22.6	1.8	1.0	6.1
December	76.6	23.4	34.4	0.9	3.1	34.6	19.6	1.1	0.8	5.5
Averages	73.3	26.7	34.7	1.8	4.8	26.7	22.6	1.2	1.1	7.1

TABLE 4.—Evaporative Tests of Moisture in Garbage.

Date.	Weight of original sample, in pounds.	Weight of dried sample, in pounds.	Water evaporated, in pounds.	Water in original sample. Percentage.	Hours drying.	Average water by months. Percentage.
Jan. 4, '06	46	10.09	35.91	78.2	88	75.9
Jan. 17, '06	50	13	37	74.0	104	
Feb. 8, '06	50	13.65	36.35	72.7	136	72.2
Feb. 26, '06	47	13.31	33.69	71.7	144	
March 23, '05	50	16	34	68.0	70	68.6
March 19, '06	49	15.09	33.91	69.2	96	
April 1, '05	40	12.5	27.5	68.7	70	69.6
April 7, '05	40	11	29	72.5	80	
April 2, '06	42	13.31	28.69	68.3	128	
April 20, '06	50	15.5	34.50	69.0	128	
May 17, '05	40	12	28	70.0	70	70.7
May 25, '05	39	12	27	69.2	60	
May 9, '06	47	13.5	33.5	71.3	104	
May 24, '06	50	14.0	36.0	72.0	130	
June 19, '05	39	8	31	79.4	60	70.6
June 9, '06	35	13.5	21.5	61.4	88	
June 25, '06	35	10.5	24.5	70.0	112	
July 2, '05	30	9.0	21	70.0	60	71.0
July 22, '05	39	11.	28	71.8	70	
July 31, '05	39	9	30	76.9	60	
July 18, '06	46	13.5	32.5	70.6	104	
July 31, '06	36	12.5	23.5	65.3	80	
August 6, '05	39	9	30	76.9	60	74.7
August 11, '05	38	11	27	71.0	60	
August 27, '05	44	11	33	75.0	60	
August 14, '06	45	11	34	75.5	112	
Sept. 4, '05	38	10	28	73.6	70	73.1
Sept. 18, '05	45	13	32	71.1	50	
Sept. 20, '04	38	9.5	28.5	75.0	90	
Sept. 8, '06	45	11.5	33.5	74.4	60	
Sept. 17, '06	44	12.5	31.5	71.6	100	
Oct. 3, '05	26	8	18	69.2	50	67.4
Oct. 17, '05	33	12	21	63.6	50	
Oct. 26, '05	29	9	20	69.0	50	
Oct. 20, '04	47	15	32	68.1	100	
Nov. 6, '05	44	13	31	70.5	88	69.2
Nov. 16, '05	37	10.92	26.08	70.5	64	
Nov. 25, '05	36.5	12.31	24.19	66.3	104	
Dec. 11, '05	45	9.84	35.16	78.1	160	.....
Average per cent. of water..	.....	.....	.....	71.4	.....	.....

4 368*D*	Rubbish.	February.	6 940	0.75	17.03	82.22	8 441	Original bulk sample reduced by quartering, paper portion pulped, coarsely ground with other rubbish, quartered, dried, pulverized, and again quartered down to test sample. Test results refer to original sample.
4 369*B*	"	February.	12 634	0.00	5.66	94.34	13 392	
4 665	"	March.	7 251	2.78	12.58	84.64	8 567	
6 280	"	May.	7 943	3.70	13.21	83.09	9 560	
7 357	"	July.	7 781	1.03	21.39	77.58	10 029	
7 358	"	July.	8 404	0.80	11.93	87.27	9 630	
7 440	"	August.	8 107	3.77	8.15	88.08	9 204	
	Average		8 437	1.83	12.85	85.32	9 889	
4 368*A*	Fine ash.	February.	0 000	3.03	80.45	16.52	.....	Same procedure as in "coal and cinders."
4 667	" "	February.	0 000	0.56	83.67	15.77	....	
	Average			1.79	82.06	16.14	.....	
4 368*C*	Clinker.	February.	0 000	1.14	89.25	9.61	.....	Same procedure as in "coal and cinders."
4 666	"	February.	0 000	0.61	90.34	9.05	.....	
	Average			0.87	89.80	9.33	.....	
4 668	Mixed refuse.	February.	3 903	0.00	46.89	53.11	7 349	Procedure as with components noted above. Total moisture not determined.
5 373	" "	April.	6 084	3.42	38.61	57.97	10 495	
6 918	" "	June.	5 700	0.31	58.95	40.74	13 991	
	Average		5 229	1.24	48.15	50.61	10 332	
7 488	Composite sample. Computed from separate tests.	See Laboratory Nos. 7 439, 7 440, 7 441.	8 254 8 452	1.18 2.26	21.35 21.14	77.47 76.58	10 654 11 037	Composed of: 53.4% Garbage, test No. 7 439. 19.6% Rubbish, " " 7 440. 27.0% Coal and cinders, test No. 7 441.

TABLE 5.—CALORIMETER TESTS. (PRELIMINARY SERIES.)

Laboratory No.	Material.	From collections in:	Test Results. Calorific power in B. t. u.	Percentage of water.	Percentage of ash.	Percentage of combustible.	Calorific power of combustibles in B. t. u. Computed.	Notes.
		1905.						
4 368*A*	Coal and cinders.	February.	4 230	1.02	61.17	37.81	11 187	
4 663	" " "	February.	3 280	1.63	52.14	46.23	7 095	
5 074*A*	" " "	March.	4 590	1.55	59.11	39.34	11 668	Original bulk sample reduced by quartering, pulverized, and again quartered down to test sample. Test results refer to original sample.
5 425*B*	" " "	April.	4 837	1.54	55.54	42.92	11 269	
6 282	" " "	May.	2 108	2.74	68.69	28.57	7 378	
7 359	" " "	July.	7 710	0.99	39.38	59.63	12 930	
7 441	" " "	August.	7 554	2.01	42.42	55.57	13 594	
	Average		4 901	1.64	54.06	44.29	11 066	
4 369*A*	Garbage.	February.	5 320	4.47	32.44	63.09	8 432	
4 664	"	February.	9 447	2.68	11.92	85.40	11 062	
5 074*B*	"	March.	7 874	0.00	12.19	87.81	8 967	Original bulk sample reduced by quartering, moisture evaporated, pulverized and again quartered down to test sample. Test results refer to practically dry garbage. For moisture in original material, see Table 4.
5 425*A*	"	April.	8 965	0.00	22.48	77.52	11 565	
6 281	"	May.	8 186	1.63	18.26	80.11	10 218	
7 356	"	July.	8 878	0.00	13.04	86.96	10 209	
7 439	"	August.	9 034	1.84	15.16	83.00	10 884	
	Average		8 243	1.52	17.93	80.55	10 233	

TABLE 6.—CALORIMETER TESTS AND

COAL AND CINDERS. (26 TESTS.)

Laboratory No.	Calorific Values. Per pound, original sample. B. t. u.	Per pound, dry sample. B. t. u.	Per pound, combustible. B. t. u.	From Collections Made: Season.	Month and year.	Proximate Analyses. Moisture. Percentage.	Volatile matter. Percentage.	Fixed carbon. Percentage.	Ash. Percentage.
921*b*	9 360	9 470	14 750	Spring ...	Early March, 1906	1.14	4.02	59.44	35.40
922*b*	8 180	8 260	14 290		Late " 1906	0.97	2.83	54.40	41.80
928*b*	8 630	8 765	15 010		Early April, 1906	1.52	4.36	53.14	40.98
929*b*	9 360	9 505	14 270		Late " 1906	1.50	3.40	62.20	32.90
936*b*	7 840	7 932	14 340		Early May, 1906	1.08	2.04	52.66	44.22
946*b*	9 645	9 850	14 400		Late " 1906	2.02	4.88	62.13	30.97
	8 836	8 957	14 504	Average, spring.....		1.37	3.59	57.33	37.71
952*b*	9 497	9 590	15 270	Summer.	Early June, 1906	2.03	4.90	56.60	36.47
971*b*	9 980	10 110	13 850		Late " 1906	1.78	5.09	66.67	26.46
833*b*	7 865	8 013	14 500		July, 1905	1.90	14.70	39.60	43.80
975*b*	7 150	7 250	14 020		Early " 1906	1.37	2.88	48.13	47.62
976*b*	9 740	9 967	14 050		Late " 1906	2.23	2.91	66.43	28.43
994*b*	9 570	9 770	14 480		Early August, 1906	2.06	6.98	59.10	31.86
995*b*	7 080	7 135	13 860		Late " 1906	0.75	3.91	47.16	48.18
	8 690	8 843	14 312	Average, summer		1.73	5.91	54.81	37.55
858*b*	8 561	8 813	12 800	Autumn.	September, 1905	2.24	7.78	59.16	30.82
998*b*	10 865	11 135	14 420		Early Sept., 1906	2.32	4.02	71.38	22.28
1 011*b*	10 585	10 840	14 980		Late " 1906	2.28	4.05	66.57	27.10
862*b*	7 805	7 920	13 580		Early Oct., 1905	1.42	1.12	56.36	41.10
875*b*	8 445	8 540	15 065		Late " 1905	1.04	2.02	54.04	42.90
882*b*	8 490	8 580	15 010		Early Nov., 1905	1.08	1.85	54.73	42.39
887*b*	7 378	7 440	15 225		Late " 1905	0.82	2.13	46.33	50.72
	8 876	9 020	14 397	Average, autumn.....		1.59	3.28	58.37	36.76
894*b*	7 270	7 320	14 460	Winter...	Early Dec., 1905	0.62	2.21	48.07	49.10
895*b*	8 210	8 265	13 810		Late " 1905	0.65	1.59	57.84	39.92
899*b*	6 940	6 980	13 450		Early Jan., 1906	0.58	1.51	50.09	47.82
900*b*	6 060	6 080	13 310		Late " 1906	0.27	2.12	43.44	54.17
901*b*	6 910	6 935	14 350		Early Feb., 1906	0.37	2.05	46.10	51.48
903*b*	6 935	6 988	13 880		Late " 1906	0.78	1.66	48.28	49.28
	7 054	7 091	13 878	Average, winter.....		0 54	1.86	48.97	48.03
	8 396	8 510	14 296	Average, all tests.....		1.34	3.73	55.00	39.93

Proximate Analyses (Welton).

Garbage. (25 Tests.)

Laboratory No.	Calorific Values. Per pound, original sample. B. t. u.	Per pound, dry sample. B. t. u.	Per pound, combustible. B. t. u.	From Collections Made: Season.	Month and year.	Proximate Analyses. Moisture. Percentage.	Volatile matter. Percentage.	Fixed carbon. Percentage.	Ash. Percentage.
921*a*	2 535	8 633	10 490	Spring	March, 1906	70.63	19.76	4.40	5.21
922*a*	2 690	8 910	9 970		Early April, 1906	69.80	22.60	4.40	3.20
928*a*	2 459	8 500	10 365		Late " 1906	71.08	19.21	4.50	5.21
929*a*	2 195	8 205	9 492		Early May, 1906	73.24	18.57	4.55	3.64
936*a*	1 779	7 025	11 500		Late " 1906	74.69	11.23	4.23	9.85
	2 331	8 294	10 273	Average, spring		71.89	18.27	4.42	5.42
946*a*	3 029	8 510	10 700	Summer	Early June, 1906	64.43	20.01	8.29	7.27
971*a*	2 238	8 235	10 440		Late " 1906	72.83	17.19	4.24	5.74
975*a*	2 329	8 560	10 550		Early July, 1906	73.43	18.11	3.97	4.49
976*a*	2 507	7 980	9 400		Late " 1906	68.59	21.38	5.30	4.73
994*a*	1 706	7 665	9 280		Early August, 1906	77.74	14.58	3.80	3.88
995*a*	2 076	8 950	10 460		Late " 1906	76.80	16.32	3.53	3.35
833*a*	1 753	8 985	10 770		July, 1905	80.50	14.68	1.59	3.23
	2 234	8 421	10 220	Average, summer		73.47	17.47	4.39	4.67
858*a*	2 190	8 533	10 550	Autumn	September, 1905	74.31	16.75	4.03	4.91
998*a*	2 063	8 720	10 520		Early Sept., 1906	76.33	16.02	3.59	4.06
1 011*a*	1 652	7 720	9 450		Late " 1906	78.60	13.97	3 52	3.91
862*a*	2 189	8 228	10 120		Early Oct., 1905	73.40	11.62	10.00	4.98
875*a*	3 050	8 805	10 720		Late " 1905	65.35	21.18	7.27	6.20
882*a*	2 354	8 003	9 820		Early Nov., 1905	70.59	18.87	5.12	5.42
887*a*	1 719	6 065	10 275		Late " 1905	71.66	11.17	5.56	11.61
	2 174	8 022	10 240	Average, autumn		72.89	15.65	5.58	5.87
894*a*	2 580	7 955	10 160	Winter	Early Dec., 1905	67.56	19.28	6.12	7.04
895*a*	1 807	8 750	10 396		Late " 1905	79.35	14.21	3.18	3.26
899*a*	2 009	9 570	11 180		Early Jan., 1906	79.01	14.28	3.67	3.04
900*a*	2 008	8 010	10 310		Late " 1906	74.93	14.84	4.63	5.60
901*a*	2 729	10 330	11 545		Early Feb.. 1906	73.60	19.84	3.78	2.78
903*a*	2 192	8 106	10 375		Late " 1906	72.95	16.54	4.58	5.93
	2 221	8 734	10 668	Average, winter		74.57	16.50	4.32	4.61
	2 233	8 351	10 338	Average, all tests		73.26	16.89	4.71	5.14

TABLE 6.—Calorimeter Tests and Prox-

Rubbish. (26 Tests.)

Laboratory No.	Calorific Values. Per pound, original sample. B. t. u.	Per pound, dry sample. B. t. u.	Per pound, combustible. B. t. u.	From Collections Made: Season.	Month and year.	Proximate Analyses. Moisture. Percentage.	Volatile matter. Percentage.	Fixed carbon. Percentage.	Ash. Percentage.
981c	7 070	7 360	8 385	Spring...	Early March, 1906	3.95	69.04	15.31	11.70
921c	6 999	7 233	9 330		Late " 1906	3.21	62.77	12.23	21.79
922c	6 395	6 555	8 692		Early April, 1906	2.46	60.94	12.64	23.96
928c	6 490	6 821	8 220		Late " 1906	4.74	64.49	14.52	16.25
929c	6 810	7 210	9 220		Early May, 1906	5.55	56.12	17.72	20.61
936c	7 220	7 690	8 370		Late " 1906	6.08	74.98	11.32	7.62
	6 831	7 140	8 682		Average, spring.....	4.33	64.72	13.96	16.99
946c	6 088	6 920	8 810	Summer.	Early June, 1906	7.13	52.60	16.52	23.75
971c	6 895	7 485	8 070		Late " 1906	7.90	70.46	15.06	6.58
975c	7 040	7 748	8 880		Early July, 1906	9.14	66.31	13.00	11.55
976c	7 028	7 463	7 950		Late " 1906	5.83	73.78	14.64	5.75
833c	6 308	6 870	7 230		July, 1905	8.20	75.60	11.70	4.50
994c	6 717	7 480	8 375		Early August, 1906	10.22	64.94	15.27	9.57
995c	6 780	7 330	8 175		Late " 1906	7.55	68.44	14.50	9.51
	6 694	7 275	8 179		Average, summer	7.99	67.45	14.39	10.17
858c	6 720	7 183	9 120	Autumn.	September, 1905	6.45	54.30	19.35	19.90
998c	5 360	5 687	7 880		Early Sept., 1906	5.74	51.84	17.20	25.22
1 011c	6 570	7 062	8 430		Late " 1906	6.97	63.06	14.91	15.06
862c	5 567	5 808	8 950		Early Oct., 1905	4.17	49.86	15.00	30.97
875c	7 130	7 575	8 170		Late " 1905	5.84	69.94	17.44	6.78
882c	7 742	8 343	8 910		Early Nov., 1905	7.16	72.68	14.26	5.90
887c	7 455	7 873	8 965		Late " 1905	5.34	67.81	15.37	11.48
	6 649	7 069	8 570		Average, autumn.....	5.95	61.36	16.22	16.47
894c	7 012	7 310	8 145	Winter...	Early Dec., 1905	4.04	71.06	15.00	9.90
895c	6 870	7 225	8 370		Late " 1905	4.90	69.07	13.06	12.97
899c	8 325	8 740	9 520		Early Jan., 1906	4.72	71.93	15.54	7.81
900c	6 250	6 510	7 985		Late " 1906	3.96	65.91	12.40	17.73
901c	7 280	7 580	8 750		Early Feb., 1906	3.88	70.50	12.75	12.87
903c	7 525	7 933	8 950		Late " 1906	5.10	68.90	15.20	10.80
	7 210	7 545	8 630		Average, winter.....	4.44	69.56	13.99	12.01
	6 832	7 251	8 503		Average, all tests.....	5.78	65.66	14.69	13.87

IMATE ANALYSES (WELTON).—(*Continued*).

OTHER TESTS.

Laboratory No.	CALORIFIC VALUES. Per pound, original sample. B. t. u.	Per pound, dry sample. B. t. u.	Per pound, combustible. B. t. u.	Material, etc.	PROXIMATE ANALYSES. Moisture, percentage.	Volatile matter, percentage.	Fixed carbon, percentage.	Ash, percentage.
923*f*	2 846	2 880	13 300	Fine ash. Screened from collections. 1905–1906.	1.20	4.02	17.38	77.40
1 031	1 578	1 583	10 038	Clinker. From collections 1905–1906.	0.32	0.73	14.99	83.96
892*e*	3 998	3 998	14 117	Ash. From practical tests in burning mixed refuse, December, 1905.	0.00	2.86	25.46	71.68
892*d*	900	900	8 730	Clinker. From practical tests in burning mixed refuse, December, 1905.	0.00	0.00	10.31	89.69
See 833*a* 833*b* 833*c*				Composite sample. Garbage.....53.4% by weight Coal and cinders. 27.0% " " Rubbish..... 19.6% " "				
833*d*	4 364	7 952	10 659	By test.	45.12	40.94		13.94
	4 294	7 823	10 613	By calculation. From Tests 833*a*, 833*b* and 833*c*.	45.11	26.63	13.83	14.43

IV.—Calorific Power of Refuse—Seasonal Variations.—Mixed refuse seemed to be an unpromising material for reduction to representative samples suitable for test in a calorimeter, but separate determinations of the components of refuse in conjunction with the mechanical analyses might be expected to serve the purpose of computing the calorific power of the refuse, within reasonable limits, for practical purposes.

Table 5 summarizes the principal features of a set of preliminary tests made at the Lederle Laboratories. The calorific determinations gave rather better results than might have been expected under the conditions of sampling then prevailing. An insufficient number of trials was made to show any seasonal variations in the heat values of the materials. Mixed refuse tests proved unsatisfactory, as might have been expected. A composite sample (No. 7488), by actual test and by computation, showed practical agreement, thus checking the assumption that the total calorific power of the refuse would equal the sum of the heat values of its constituents.

In February, 1907, there became available a very comprehensive series of calorific determinations, made during 1905 and 1906 at the laboratory of the Commissioners of Accounts, under the direction of Mr. Otto H. Klein, Chief Engineer, by B. F. Welton, Assoc. M. Am. Soc. C. E. Large amounts of the several components of refuse, from the mechanical analyses for each month, were reduced by quartering, moisture was evaporated from "garbage," and the different materials were roughly pulverized to representative samples at the West New Brighton crematory. Further preparation of samples for test in a Mahler bomb calorimeter was made at the laboratory. The method of procedure in these determinations differed from that of the preliminary determinations in the case of "rubbish," where the paper content was not pulped before being pulverized, also with "fine ash" and "clinker," which were burned with naphthaline to effect complete oxidation of the relatively small percentage of combustible matter.

The results of the various calorific determinations and proximate analyses by Mr. Welton, with the average seasonal variations of the different components, are shown in Table 6.

In this summary "coal and cinders" has a lower value in winter than during other seasons of the year, probably owing to the more complete combustion of fuel in cold weather; "garbage" shows no

FIG. 1.—BERMONDSEY DESTRUCTOR IN REAR OF TOWN HALL.

FIG. 3.—SHEERNESS DESTRUCTOR. A SCHOOL HOUSE IS ONLY 8 FT. AWAY FROM THE REFUSE STORAGE ROOM.

FIG. 2.—ANOTHER VIEW, SHOWING LOCATION, BERMONDSEY DESTRUCTOR.

FIG. 4.—VIEW FROM WATER TOWER AT SHEERNESS, SHOWING SURROUNDINGS OF DESTRUCTOR.

appreciable seasonal variation, and "rubbish" has a higher power in winter than in spring, summer, or autumn. The test of a single average sample of "fine ash" and of "clinker" leaves the question of seasonal variations undetermined with regard to these materials. On the whole, the seasonal differences are not marked, except in the case of "coal and cinders." The columns giving the heat units per pound of dry sample show the variations in the mechanical analyses and the bulk sampling, while the calorific power per pound of combustible compares the character of the materials tested, and was used as an additional check in the calorimeter determinations. The results of the tests shown in Table 6 were much more consistent than was ever hoped for.

Based on the composition of refuse (Table 3) and the final calorific tests (Table 6), the heat value per pound of refuse for different periods has been deduced, as shown in Table 7.

TABLE 7.—Calorific Value per Pound of Refuse for Different Periods.

Period.	Calorific power of combustible. B. t. u.	Moisture, per cent.	Ash, per cent.	Combustible, per cent.	Remarks.
Spring.....	4 747	14.03	50.06	35.91	Computed results based on average figures for corresponding periods in Tables 3 and 6, except that average calorific values for summer components were used in arriving at September results.
Summer....	3 477	28.86	39.74	31.40	
Autumn.....	3 833	27.74	39.74	32.52	
Winter.....	4 358	13.11	52.72	34.17	
Year.....	4 274	19.74	46.03	34.23	
September..	3 265	35.83	33.69	30.48	

Compared with the results computed from the preliminary calorific tests (Table 5), the values in Table 7 will be found to be considerably higher, due to including the combustible power of the "fine ash" and the greater heat value of "coal and cinders."

September has the minimum calorific power and the maximum amount of moisture, thus becoming the most trying month in burning mixed refuse.

V.—Practical Tests in Burning Refuse.—While other examinations were progressing, practical tests in burning refuse were conducted at

TABLE 8.—PRACTICAL TESTS

Test No.	Date, 1905.	Kind of material burned.	Amount of refuse, in pounds.	Time burning, in minutes.	Rate of burning, per square foot of grate, per hour.	Temperature or appearance of fire.	Air pressure in ash-pit. Inches of water.	Chimney smoke.
1	Apr. 16.	Composite sample "*A*".....	500	51	49	Short orange to yellow flame.	.....	Hardly visible except when feeding.
2	"	Composite sample "*B*"....	500	40	62½	" " "	.....	" " "
3	"	Mixed refuse as collected..	1 000	20	.....	Smoke. No flame. Fire smothered.	.....	.....
4	"	" "	1 000	60	83.3	Short yellow to white flame.	.....	Hardly visible except when feeding.
5	Apr. 28.	" "	1 600	90	88.8	Short yellow to white flame, 800 to 1 200° fahr.	1½	Dense white at start. Hardly visible later.
6	Apr. 30.	" "	470	37	63.5	800-1 200° fahr. Melted lead, zinc and aluminum.	¾	Light white smoke.
7	"	" "	470	50	....	Fire failed to spread.	¾	.....
8	May 28.	" "	500	37	67.5	800° fahr.	¾	Light white smoke.
9	"	Composite sample "*C*"....	1 000	90	55.5	1 200° fahr.	¾	Dense to light white smoke.
10	June 25.	Mixed refuse as collected..	1 100	90	61	Yellow to white flame.	1½	" " "
11	July 9.	" "	.....	.....	.....	.....	.....	Murky smoke at start, then light white.
11		Material in rain overnight. Contained 37.5% moisture.	.....	.....	.....	.....	.....	.....
12	July 23.	Mixed refuse as collected..	950	60	79	800°-1 200° fahr.	1	Light white to hardly visible smoke.
13	Aug. 6.	Composite sample "*D*".....	500	.....	.....	Clear hot fire.	¾	Light white smoke.
14	"	" " "*E*".....	300	.....	.....	Fire smothered.	1½	" " "
15	Aug. 20.	Mixed refuse as collected..	1 500	90	83.3	Clear hot fire.	1½	Murky at start. Hardly visible later.
16	Sept. 10.	" "	1 431	.....	.....	Poor fire. Low temperature.	1	Murky to white dense smoke.
17	Sept. 17.	" "	1 126	60	94	Low temperature.	1¼	Murky to light white.
18	Sept. 24.	" "	986	60	82	1 000° fahr.	¾	Dense to light white smoke.
19	Oct. 1.	" "	1 547	90	86	Above 1 200° fahr.		Light white to hardly visible.

IN BURNING REFUSE.

Residue (excluding dust).							
Clinker.		Ashes.		Total residue.			
Weight, in pounds.	Percentage.	Weight, in pounds.	Percentage.	Weight, in pounds.	Percentage.	Character of clinker.	Notes.
179	35.8	.....	.....	.....	.....	Vitreous.....	Refuse ignited on bed of hot coals. Two charges of 250 lb. each. Burned freely.
231	46.2	.....	.....	.....	.....	"	Hot furnace walls. Material burned freely.
.....	.....	.....	.....	.....	.....	.....	Refuse not properly ignited. Too much air from blast.
301	30.1	.....	.....	.....	.....	Vitreous. Well burned.	Same material as in 3. Started properly. Burned freely.
379	23.7	107	6.7	486	30.4	" "	Material burned freely.
137	29.2	.....	.....	.....	.....	" "	Material burned freely. One-half of grate clinkered, other half of clinker spread to start Test 7.
.....	.....	.....	.....	.....	.....	.....	Clinker from former test failed to ignite fresh refuse.
140	28	43	8	183	36.6	Vitreous.....	Material burned freely.
329	33	87	9	416	42	Dense vitreous clinker.	Hot furnace walls from Test 8. Material burned freely. Clinker very hard and dense.
378	34.4	.....	.....	.....	.....	Vitreous.	Test to determine amount of refuse that could be burned before clinkering became necessary. Result—about 80 lb. per sq. ft. of grate.
.....	.....	.....	.....	.....	.....	.....	Test to determine amount of kindlings necessary to ignite the refuse properly. First—20 lb. of paper was tried with partial success.
.....	.....	.....	.....	.....	.....	.....	Second—11 lb. paper, 59½ lb. wood used. Material burned freely.
.....	.....	.....	.....	.....	.....	.....	Much smoke in furnace room during both trials—due to moisture in refuse.
269	28.3	.....	.....	.....	.....	Vitreous	Refuse in rain over-night. Contained large quantity of garbage, grass, weeds and paper. Very little ashes.
.....	.....	.....	.....	.....	.....	.....	Burned freely. Clinker apparently very dense.
.....	.....	.....	.....	.....	.....	.....	Charged on Test 14 when "*D*" was burned down to glowing mass. Fire smothered by sweepings in "*E*."
477	31.8	136	9.0	613	40.8	Vitreous	Refuse mostly green vegetable matter with small amount of ashes and rubbish.
.....	.....	.....	.....	.....	.....	No clinker formed.	Refuse contained much wet sawdust, decayed fish and vegetables from stores. Moisture 56.5%. Not completely burned. Very smoky in furnace room.
317	28.1	.....	.....	.....	.....	Fair. Not dense. Material not all burned.	Large amount of yard sweepings in refuse, very little combustible. Too much air from blower.
264	26.8	.....	.....	.....	.....	Small amount of clinker. Mostly ashes on grate.	Large quantity garbage, small amount of ashes and rubbish. Less air than in 17 with better results. Not much clinker formed.
379	24.5	125	8	504	32.5	Vitreous. Well burned.	Material from residential section. Large amount of ashes.

TABLE 8.—PRACTICAL TESTS

Test No.	Date. 1905.	Kind of material burned.	Amount of refuse, in pounds.	Time burning, in minutes.	Rate of burning, per square foot of grate per hour.	Temperature or appearance of fire.	Air pressure in ash-pit. Inches of water.	Chimney smoke.
20	Oct. 15.	Mixed refuse as collected..	1 495	80	93	Above 1 200° fahr.	1½	Light white to hardly visible.
21	Oct. 29.	" "	1 254	80	78.4	" " "	1¼	" " "
22	Nov. 12.	" "	1 148	120	47.8	" " "	1	" " "
23	Nov. 26.	" "	1 159	90	64.4	" " "	1	" " "
24	Dec. 10.	" "	1 559	125	62.4	" " "	1½	Murky at start. Hardly visible later.
	1906.							
25	Jan. 7.	" "	1 200	90	66.6	Temp. low.	Gauge broken.	Light white smoke.
26	Jan. 21.	" "	1 128	60	94	Temp. low.	"	Murky at start. Hardly visible later.
27	Feb. 4.	" "	1 120	70	80	Above 1 000° fahr.	"	Light white at start. Not visible later.
28	Feb. 18.	" "	1 180	90	74	Above 1 200° fahr.	"	Murky at start Hardly visible later.
29	Mar. 11.	" "	1 500	90	83.3	Temp. by pyrometer 1 600° fahr.	"	Light white at start. Hardly visible later.
30	Mar. 25.	" "	1 000	90	55.5	Combustion chamber temp. by pyrometer 1 100° fahr.	"	Light white to hardly visible.
31	Apr. 8.	" "	1 335	90	74.2	Highest temp. 1 420° fahr.	"	White at start. Not visible later.
32	Apr. 22.	" "	1 200	75	80	Highest temp. 1 400° fahr.	"	Light white at start. Not visible later.
33	May 6.	" "	1 200	75	80	Highest temp. 1 350° fahr.	"	Light white at start. Hardly visible later.
34	May 20.	" "	1 200	60	100	Highest temp. 1 150° fahr.	"	Dense white at start.
35	June 3.	" "	1 100	75	73.3	Highest temp. 1 320° fahr.	"	Light white smoke.
36	June 17.	" "	1 280	70	91.4	Highest temp. 1 400° fahr.	"	Light white at start. Not visible later.
37	July 8.	" "	1 880	.....	.....	Highest temp. 1 400° fahr.	"	Light white at start.
38	July 22.	" "	1 400	.....	.....	Highest temp. 1 300° fahr	"	Heavy white at start.
39	Aug. 5.	" "	1 250	115	.....	Highest temp. 1 400° fahr.	"	Dense white at start. Light white later.
40	Aug. 19.	" "	1 100	60	92	Temp. high. 1 700° fahr.	"	Light white smoke.
41	Sept. 2.	" "	1 030	60	86	Highest temp. 1 350° fahr.	"	Dense white at start.
42	Sept. 23.	" "	1 200	75	80	Highest temp. 1 400° fahr.	"	Dense white at start. Hardly visible later.

IN BURNING REFUSE—(*Continued.*)

RESIDUE (EXCLUDING DUST).

Clinker.		Ashes.		Total residue.			
Weight, in pounds.	Percentage.	Weight, in pounds.	Percentage.	Weight, in pounds.	Percentage.	Character of clinker.	NOTES.
326	21.8	90	6	416	27.8	Vitreous. Well burned.	Large quantity of green vegetables and decayed fish. Burned rapidly.
327	26	73	6	400	32	" " "	Large proportion of fine ash and vegetables, very little rubbish.
432	37.6	82	7.2	514	44.8	" " "	Large amount of garbage and fine ash.
334	28.8	87	7.5	421	36.3	" " "	Large proportion of fine ash, small amount of coal and cinders.
564	36.2	181	11.6	745	47.8	" " "	About half of refuse burned down, mass clinkers then removed and other half of refuse charged on coals of first fire.
							Material burned freely in both cases.
						Poor. No mass clinker. Mostly ashes.	Poor quality of material. Large amount of incombustible. Residue not completely burned.
							Material poor in quality.
						Rather poor.....	Purpose of test—to determine the amount of waste heat in clinker. 511 B. t. u. per lb.
							Material of fair quality.
						Hard. Well burned.	Waste heat in clinker, tested, gave 315 B. t. u. per lb.
367.5	31.1	141	12.0	508.5	43.1	Hard. Well burned.	Refuse of fair quality. Waste heat from residue on grate gave 488 B. t. u. per lb.
518	34.5	132	8.8	650	43.3	Hard. Well burned.	Bristol electric pyrometer used for taking temperature readings. Combustion chamber 1 600° F. Chimney 1 000°
478	47.8	80	8.0	558	55.8	Hard. Well burned.	Fire not well started. Excess of air.
421	31.5	88	6.6	509	38.1	Hard. Well burned.	Material wet. In rain over-night. Furnace room smoky.
290	24.2	161	13.4	451	37.6	Hard. Well burned.	Refuse dry, large quantity of fine ash.
293	24.4	100	8.3	393	32.7	Hard. Well burned.	Refuse damp. Burned rapidly.
302	25.2	63	5.2	365	30.4	Poor. Little mass clinker.	Fire poorly started. Much smoke from furnace. Excess of air.
219	19.9	115	10.5	334	30.4	Fair.....	Excess of air.
308	24.1	115	9.0	423	33.1	Hard. Well burned.	Material burned rapidly. Probable excess of air.
							Electric motor broke down. Test not finished. Material of very poor quality—very wet.
							Test not finished. Belt to fan broke. Material in rain over-night.
							Material practically all garbage, contained 40% water. Required coal to burn it properly.
259	23.5	108	9.8	367	33.3	Well burned. Not much mass clinker.	Material burned freely. High tem. Clinker cooled in air dropped 1 360° fahr. in 40 min.
212	20.6	57	5.5	269	26.1	Not much lump clinker.	Some unburned corncobs in ashes. Clinker cooled by fan blast dropped 1 280° fahr. in 20 min. Excess air.
272	22.7	73	6.0	345	28.7	Not much lump clinker. Not completely burned.	Unburned corncobs and wood. Excess of air. Test to determine temp. to which air could be heated by hot clinker. See Note 42.

TABLE 8.—PRACTICAL TESTS IN BURNING REFUSE.—(*Continued.*)

Composition of Refuse Burned.

Test No. 1. Composite "A."

Coal and cinders	208 lb.	= 41.6%
Rubbish	31½ "	= 6.3 "
Clinker	45½ "	= 9.1 "
Garbage	194 "	= 38.8 "
Glass, metals, etc.	21 "	= 4.2 "
Totals	500 "	100 "

Test No. 2. Composite "B."

Coal and cinders	145 lb.	29%
Rubbish	20 "	4 "
Clinker	35 "	7 "
Garbage	265 "	53 "
Glass, metals, etc	35 "	7 "
Totals	500 "	100 "

Tests 3 and 4. Sample.

Coal and cinders	60 lb.	37.5%
Garbage	13 "	8.1 "
Rubbish	4 "	2.5 "
Glass, metals, etc.	3 "	1.9 "
Clinker	7 "	4.4 "
Fine ash	73 "	45.6 "
Totals	160 "	100 "

Test 5. Sample.

Coal and cinders	68 lb.	46 3%
Garbage	17 "	11.5 "
Rubbish	11.5 "	7.8 "
Glass, metals, etc	4.5 "	3.0 "
Clinker	5 "	3.4 "
Fine ash	41 "	28.0 "
Totals	147 "	100 "

Moisture test, 112 hr. drying.

Weight, original sample	56 lb.
" dried "	45 "
Water evaporated	11 "

Moisture, 19.6%.

Tests 6 and 7. Sample.

Coal and cinders	11.5 lb.	27.0%
Garbage	10 "	23.6 "
Rubbish	2 "	4.7 "
Glass, metals, etc.	1.5 "	3.5 "
Clinker	0.5 "	1.2 "
Fine ash	17 "	40.0 "
Totals	42.5 "	100 "

Moisture test, 112 hr. drying.

Weight, original sample	42.5 lb.
" dried "	33.5 "
Water evaporated	9.0 "

Moisture, 21%.

Test 8. Sample.

Fine ash	18 lb.	31%
Coal and cinders	13 "	22 "
Rubbish	6 "	10 "
Garbage	16 "	28 "
Glass, metals, etc.	5 "	9 "
Totals	58 "	100 "

Moisture test, 112 hr. drying.

Weight, original sample	42 lb.
" dried "	35 "
Water evaporated	7 "

Moisture, 16.6%.

Test 9. Composite "C."

Coal and cinders	243 lb.	24.3%
Clinker	81 "	8.1 "
Glass, metals, etc	19 "	1.9 "
Garbage	480 "	48.0 "
Rubbish	177 "	17.7 "
Totals	1 000 "	100 "

Test 10. Sample.

Coal and cinders	27 lb.	22.7%
Rubbish	9 "	7.6 "
Glass, metals, etc.	6 "	5.0 "
Garbage	23 "	19.3 "
Clinker	4 "	3.4 "
Fine ash	50 "	42.0 "
Totals	119 "	100 "

Sample of material from Test 10 showed 5 700 B. t. u. on dry refuse. (See Table 5, Lab. No. 6 918.)

Test 11. Moisture Test.
96 hr. drying.

Weight, original sample	32 lb.
" dried "	20 "
Water evaporated	12 "

Moisture, 37.5%.

TABLE 8.—PRACTICAL TESTS IN BURNING REFUSE.—(*Continued.*)

Composition of Refuse Burned.

Test 14. Composite "E."

Coal and cinders	60.75 lb.	12.1%
Clinker	20.25 "	4.1 "
Glass, metals, etc	4.75 "	1.0 "
Garbage	120.00 "	24.0 "
Rubbish	44.25 "	8.8 "
Street sweepings	250.00 "	50.0 "
Totals	500 "	100 "

Test 13. Composite "D."

Coal and cinders	121.5 lb.	24.3%
Clinker	40.5 "	8.1 "
Glass, metals, etc	9.5 "	1.9 "
Garbage	240.0 "	48.0 "
Rubbish	88.5 "	17.7 "
Totals	500 "	100 "

Test 24. Sample.

Fine ash	52.0 lb.	45.8%
Clinker	7.0 "	6.2 "
Glass, metals, etc	4.5 "	3.9 "
Coal and cinders	33.5 "	29.5 "
Garbage	13.0 "	11.5 "
Rubbish	3.5 "	3.1 "
Totals	113.5 "	100 "

Test 16. Moisture Test.
96 hr. drying.

Weight, original sample	23 lb.
" dried "	10 "
Water evaporated	13 "

Moisture, 56.5%.

Test 32. Moisture.

Weight, original sample	23.5 lb.
" dried "	21.0 "
Water	2.5 "

Moisture, 10.7%.

Test 34. Sample.

Fine ash	23 lb.	29.1%
Coal and cinders	28 "	35.4 "
Clinker	4 "	5.1 "
Rubbish	8 "	10.1 "
Garbage	16 "	20.3 "
	79 "	100 "

Moisture test.

Original sample	32 lb.
Dried "	26 "
Water	6 "

Moisture, 19%.

Test 35. Moisture.

Original sample	35 lb.
Dried "	29 "
Water	6 "

Moisture, 17.2%.

Test 38. Moisture.

Original sample	35 lb.
Dried "	28 "
Water	7 "

Moisture, 20%.

Test 36. Moisture.

Original sample	35 lb.
Dried "	28 "
Water	7 "

Moisture, 20%.

Test 40. Estimated composition.

Ashes	60% by weight.
Garbage	30 " " "
Rubbish	8 " " "
Tins, etc	2 " " "

Note.—Hot clinker from grate cooled 1 300° fahr. in air, in 40 min.

Test 39. Moisture.

Original sample	30 lb.
Dried "	18 "
Water	12 "

Moisture, 40%.

Test 41. Estimated composition.

Ashes	50% by weight.
Garbage	30 " " "
Rubbish	15 " " "
Tins, etc	5 " " "

Note.—35 lb. hot clinker was cooled from 1 700° fahr. to 420° fahr. (1 280° fahr.) by air blast in 20 min.

Test 42. Estimated composition.

Garbage	60% by weight.
Ashes	30 " " "
Rubbish	7 " " "
Tins, etc	3 " " "

Note.—Air at 84° fahr. was heated to 423° in passing through hot clinker. After 5 min. the air forced through the hot clinker by the fan dropped from 423° to 270° fahr. 74 lb. of clinker with large volume of air was used.

TABLE 8.—Practical Tests in Burning Refuse.—(*Continued*).

Summary Showing Residue from Tests.

Season.	Refuse burned, in pounds.	Time of burning, in minutes.	Rate of burning, per square foot of grate per hour.	Residue.			
				Clinker.		Ashes from Ash-pit.	
				Pounds.	Percent.	Pounds.	Percent.
Spring.....	9 535	607	78.5	2 821	29.6	774	8.1
Summer.....	4 980	295	84.5	1 263	25.4	474	9.5
Autumn.....	8 833	595	74.0	2 282	25.8	587	6.7
Winter.....	2 739	215	63.5	932	34.0	322	11.7
Year.....	26 087	1 712	76.0	7 298	28.0	2 157	8.3

the West New Brighton crematory. The coal grate of the Dixon garbage furnace (12 sq. ft. in area) was used for burning the materials. Air at atmospheric temperature was forced into the ash-pit by a Sturtevant fan driven by an electric motor. Paper and wood, reduced to a glowing mass on the grate, were used to start the fire, except where otherwise noted in Table 8, which summarizes the results of forty-two trials, with other experimental data. In spite of such adverse conditions as cold air blast, cold furnace walls, leakage of air through warped doors, and high percentage of moisture, with small amount of combustible in some trials, all the tests except one were successful in destroying mixed household refuse, though unburned particles were at times found in the residue.

The general deductions from these rough practical tests may be summed up as follows:

1.—Household refuse, as collected in this district, when burned in a properly designed furnace, will be self-combustible, under ordinary conditions, showing higher calorific power in winter than in summer. Screened refuse will give better results in burning than unscreened.

2.—About 80 lb. of refuse per sq. ft. of grate could be burned before it became necessary to remove the clinker.

3.—The process may be made continuous by retaining the heated coals from the top portion of the fire and removing the mass clinker. Coal may be required to heat the furnace walls if the operation of the plant is not made continuous.

4.—The rate of burning will be higher in summer than in winter.

FIG. 1.—VIEW SHOWING LOCATION OF DESTRUCTOR AT WREXHAM.

FIG. 3.—RATHMINES DESTRUCTOR, IRELAND. VIEW OF PLANT AND SURROUNDINGS FROM CLOCK TOWER OF TOWN HALL.

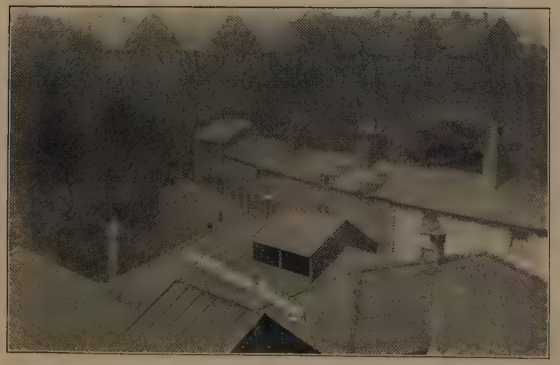

FIG. 2.—VIEW FROM WATER TOWER AT WREXHAM, SHOWING HOUSES NEAR DESTRUCTOR BUILDING.

FIG. 4.—MOSS SIDE DESTRUCTOR, MANCHESTER. HOUSES IN BACKGROUND WERE ERECTED AFTER DESTRUCTOR WAS PUT IN OPERATION.

5.—The percentage of clinker will also vary with the seasons, being high in winter and low in summer. The total residue was not determined, as a large portion of the fine ash was carried over by the air blast and could not be recovered.

6.—The heat lost by the removal of hot clinker varied from 300 to 500 B. t. u. per lb. of clinker.

7.—Street sweepings from this locality could not be burned with household refuse, except when mixed in small proportions.

VI.—Power from Refuse—Temperatures.—In spite of the fact that British engineers have had years of experience with refuse destructors, the writer has been able to find only one scientific test which would assist in determining the actual power to be obtained from refuse of known calorific value. Mr. C. E. Stromeyer (in January, 1903), by continuous gas analysis, made an economic test and worked out a heat balance for the destructor at Nelson, England. Guided by some of the results of this test, and based upon the experience gained and experiments made in the burning of local mixed refuse, the writer has arranged the approximate heat balance shown in Table 9.

Column 12 in Table 9 answers the question (approximately and theoretically) as to the probable evaporative power of local refuse when burned in an up-to-date destructor, and Column 13 shows the estimated temperature of the combustion chamber under the assumed conditions of burning.

The calculated evaporative power of local refuse is generally higher, and shows a greater seasonal variation than that indicated by published tests with British refuse. September remains the critical month in destroying the material.

Summarizing the results of examinations, tests, and experiments with mixed household refuse from the district considered, the following conclusions are derived:

1.—Average local refuse differs mainly from what is known concerning average English refuse in the higher percentage of incombustible matter and the lower percentage of water. The average results to be expected in power production are surprisingly high, and the seasonal variations are greater with local refuse than with British refuse.

2.—Under expert management, with a properly designed furnace, the process can be carried out in settled communities without nuisance.

TABLE 9.—APPROXIMATE HEAT BALANCE PER POUND OF REFUSE. ESTIMATED TEMPERATURES.

(1)	(2)	Losses due to:								(12)	(13)
Period.	Calorific power of refuse, in B. t. u.	(3) Moisture, in B. t. u.	(4) Heat in dry chimney gases, in B. t. u.	(5) Unburned carbon in clinker, in B. t. u.	(6) Unburned carbon in ashes, in B. t. u.	(7) Heat in clinker, in B. t. u.	(8) Forced draft, in B. t. u.	(9) Radiation, etc., in B. t. u.	(10) Total losses, in B. t. u.	(11) Net useful heat to boiler, in B. t. u.	Equivalent evaporation, from and at 212° fahr. (useful steam), pounds of water.	Estimated temperature of combustion chamber, in degrees Fahrenheit.
Spring	4 747	184	465	266	324	55	121	949	2 364	2 383	2.46	2 370
Summer	3 477	373	407	229	380	38	106	695	2 228	1 249	1.29	1 710
Autumn	3 883	363	421	232	208	44	110	767	2 205	1 678	1.68	1 950
Winter	4 358	174	443	306	468	63	115	872	2 441	1 917	1.98	2 140
Year	4 274	259	444	252	332	54	116	855	2 312	1 962	2.03	2 150
September	3 265	465	395	229	380	54	103	653	2 279	986	1.02	1 550

FIG. 1.—BROMLEY DESTRUCTOR. HOUSES TO THE RIGHT.

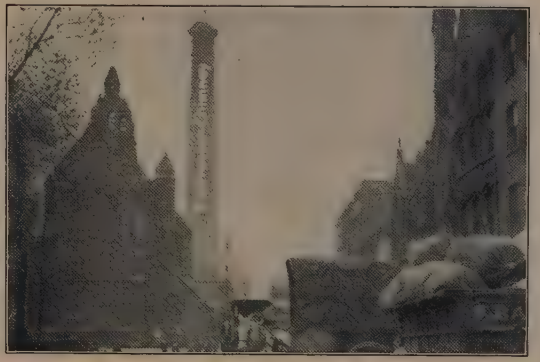

FIG. 3.—SHOREDITCH DESTRUCTOR IS IN REAR OF CHAPEL, WITH BUILDINGS ON ALL SIDES.

FIG. 2.—BROMLEY DESTRUCTOR. RESIDENCES NEARBY—IN PLAIN SIGHT OF PLANT.

FIG. 4.—ANOTHER VIEW AT SHOREDITCH. THE DESTRUCTOR IS AT THE RIGHT OF THE CHIMNEY.

3.—The average local residue will be greater than the average English residue mainly because of the high percentage of fine ash which will to some extent be carried away from the fire-grate by the forced draft.

4.—As compared with the local cost of burning garbage and caring for "ash and rubbish" dumps, the cost of the destruction of mixed refuse will probably be higher, though a proper utilization of the steam generated and the clinker resulting may offset this increase in cost, while a rearrangement of the refuse collection system may tend further to make the cost of the methods comparable.

5.—For the particular conditions herein considered, mixed-refuse destruction appears to offer the best solution of the problem.

Part II.

Pending the acquisition of property for the erection of the first installation, the writer was directed by the President of the Borough of Richmond to examine a number of British refuse destructors in actual operation, so that the various reports might be verified and the differences in composition, quantity, and quality of refuse noted, in addition to obtaining data whereby the best type of modern destructor could be secured. Particular instructions were given to observe the weak points in mixed-refuse destruction.

During May and June, 1906, thirty-nine installations in Great Britain were inspected, and in August, 1906, the only destructor of the British type in the eastern part of North America, at Westmount, a suburb of Montreal, Canada, was visited in company with Louis L. Tribus, M. Am. Soc. C. E., Consulting Engineer to the President of the Borough of Richmond. Thus forty destructors were examined, thirty of which were in England, three in Wales, three in Ireland, three in Scotland, and one in Canada.

In covering a large field, wherein variations in type and design of destructors burning different kinds of refuse were encountered, a better general knowledge of the results attained in disposing of refuse was secured than if attention had been concentrated on a smaller number of plants.

Efforts were made to obtain complete data following a schedule covering the main factors in the process of mixed-refuse destruction, so that the various features of each installation might be tabulated

and compared with other plants visited. The facts secured at the different destructors were checked by comparison with published data where such were available.

Table 10 represents in condensed form the results of the writer's observations on the forty destructors examined. In summarizing and discussing British practice in refuse collection and destruction, the general order of the headings in Table 10 will be followed.

The Collection of Refuse.—British household refuse, consisting of ashes, garbage, and rubbish, is thrown into one can, bin, or ash-pit by the householder. The materials are then dumped or shoveled into a wagon or cart and removed to the place of final disposition. Organic waste (garbage) is not separated from ashes and rubbish, as is usually the practice in the large cities of the United States, where the reduction system has been generally adopted.

Municipal ownership and operation of the refuse-collection service in Great Britain is general, and the contract system is almost unknown. Single-horse covered wagons or vans, holding about 3 cu. yd. of material, are common in the larger cities, and the type of horse used is rather superior to any noticed in the vicinity of New York, on similar work.

Frequency of Collection.—Refuse is collected weekly in Great Britain, as a rule, though in some municipalities the material is removed daily, while in other cities, where the old-style ash-pit system is in use, a monthly clean-out is common. To an American acquainted with the daily collection of garbage in the large cities of the United States, this delay in removal may seem to be unsanitary, especially in summer, when organic wastes decompose rapidly, but differences in conditions explain the matter:

First.—Ashes, garbage and rubbish are thrown together into one receptacle, and the ashes absorb the excess of water and tend to deodorize or retard the decomposition of the garbage.

Second.—The average Briton is not as wasteful as the average inhabitant of Richmond Borough, as the figures in Table 11 indicate.

Thus the period during which the ordinary refuse receptacle overflows will be longer in Great Britain than in the West New Brighton District, for which accurate figures have been compiled.

Third.—Differences in climate and in the habits of the people may explain further the long-time period of refuse collection practiced in Great Britain.

TABLE 11.

Locality.	Tons (of 2 000 lb.) per 1 000 inhabitants per day.	Authority.
Average, England	0.83	George Watson.
South of England	0.56 to 0.84	H. N. Leask.
Middle England	1.12	H. N. Leask.
Northeastern England	2.24	H. N. Leask.
Tottenham, England	0.63	J. E. Butler-Hogan, M. D.
Bradford, England	1.176	Ernest Call.
Glasgow, Scotland	0.86	D. McColl.
Dublin, Ireland	0.836	F. J. Allen.
Richmond Borough, N. Y.		
Average	1.59	Author.
Maximum (winter-spring)	1.94	Author.
Minimum (summer)	1.18	Author.

The Cost of Collection.—English municipalities generally have excellent accounting systems, but poor cost-keeping methods, though in some cases both accounting and cost-keeping are in advance of most American cities.

Figures on the cost of collection of refuse (which includes removal to the place of final disposition) were available at four places, as follows:

Tottenham, England	$0.56	per ton of 2 000 lb.
Swansea, Wales (about)	0.67	" " " " "
Glasgow, Scotland (one year)	0.63	" " " " "
Bradford, England (one year)	0.68	" " " " "
Average	$0.635	" " " " "

In a later paragraph, the average labor cost for destroying mixed refuse in some British destructors is shown to be $0.215, hence the cost of collecting the material is about three times the cost of destroying it, when operating expenses alone are considered. Thus, in any system of final disposition of refuse, the cost of collection becomes a most important factor in the economy of the method.

Composition of Refuse.—The word refuse, used in this portion of the paper, refers to household refuse alone, as street sweepings were not burned at any of the plants visited. In some special cases, however, night-soil and dead animals were cremated at the destructors.

Household refuse will vary in composition according to:

(a).—The geographical position of the city and its relation to the fuel and food supply;

(b).—The particular district of the city considered, as business, residential, tenement, or manufacturing;

(c).—The character and habits of the people;

(d).—The season of the year.

Very little attention has been given by municipal engineers to determining the composition and calorific value of British refuse, and so only scattered data are available for comparison with the refuse of other countries. British destructor makers, however, base their guaranties of power production upon an examination of the refuse in each locality. This involves a previous experience with results obtained under like conditions at other places. Thus a sight valuation or estimate of the composition of refuse in different cities should be of service in determining the feasibility of burning refuse without the use of additional fuel, and for comparing results in power production.

At each plant an estimate, by weight, was made of four general components of refuse, *viz.:* (1) ashes, including other small inorganic materials; (2) garbage, or organic matter; (3) rubbish, or paper, straw and such light combustible materials; and (4) glass, metal, etc. Some previous experience helped the writer in this respect, though it should be understood that the figures for the different cities shown in Table 10 are based upon the quantity of refuse on hand when each plant was visited, and are merely rough estimates.

At first sight, the difference in general appearance of British refuse and that found in the vicinity of New York was apparent. The color of British refuse is dark brown to black, due to the soft coal ashes, while ashes from hard coal, such as used in New York, are light gray in color. Again, British refuse varies in appearance and composition with the distance from the coal fields, the character of the district, and the habits of the people. In London and the south of England, the refuse is rather lighter in color than that found in Middle England in or near the colliery districts, where it is darker, and more unconsumed lump coal is visible.

In comparing American household refuse with the estimates given in Table 10, localities having the same general characteristics and for the same period of the year should be chosen. On this basis, as a general conclusion, the writer is of the opinion that British refuse

contains more ashes, less garbage, less rubbish and more moisture than household refuse in the vicinity of New York, though the higher percentage of moisture during May and June, 1906, might have been due to a prolonged rainy period. From such information as was obtainable regarding the composition of British refuse, it would appear that no such seasonal variations occur as may be found in comparing American summer with American winter refuse, while, during the fruit season, British refuse contains no wastes comparable to melon rinds and corn cobs.

A superficial observation of British refuse is apt to prove deceptive in the amount of garbage present in the mixed mass, as the dark ash tends to cover and conceal garbage which would otherwise be quite apparent in the lighter colored anthracite ash.

Apparent Value as a Fuel.—That British refuse has a fuel value is proved beyond a doubt by the two hundred or more destructors in which refuse is burned throughout the year without additional fuel. There would seem to be no large seasonal variation in the calorific power of the material, though in one instance a difference in steam production of 15% less in summer than in winter has been noted.

The average evaporation for eighteen tests quoted in Table 10 amounts to 1.62 lb. of water per pound of refuse. Assuming an efficiency of 50% (boiler and grate) per pound of refuse, the calorific value of the average material burned during the tests would amount to 3 130 B. t. u., which practically agrees with the estimated calorific power of average British refuse given by various authorities on the subject.

Location of Plants.—The location of a plant for the final disposition of refuse has a most important bearing on the cost of the collection (including removal) of the material. Economy in collection requires that the plant shall be centrally located with regard to the district served, and that loaded collection wagons or carts shall proceed with the road gradient.

Of the forty destructors, four were critically located, so that the least nuisance would probably result in the abandonment of the plants; seventeen were centrally located in advantageous positions with regard to the district served, but the surrounding houses were not in close proximity to the destructors; nineteen were placed on the outskirts of towns and not likely to cause complaint, even if the plants were not well operated.

Complaints of nuisance due to the location of British refuse destructors in settled localities are said to be rare, and, as far as could be determined, very few of the plants visited deserved condemnation in this respect. The photographs which accompany this paper indicate beyond a doubt that mixed-refuse destructors can safely be placed in central localities.

Certainly the town councilmen of Bermondsey Borough, London, would not allow a nuisance directly in the rear of the Town Hall (Figs. 1 and 2, Plate XXXVIII), nor would the people living about the Sheerness destructor (Figs. 3 and 4, Plate XXXVIII) permit an ill-smelling plant to continue in existence. At Wrexham, in Wales, (Figs. 1 and 2, Plate XXXIX), the destructor is critically situated, and at Rathmines, just outside Dublin, Ireland, the destructor is in the rear of the Town Hall (Fig. 3, Plate XXXIX), with houses nearby. In many other cases destructors are located so that any nuisance would certainly result in complaints by people living in the vicinity, and probably end by closing the plant.

It is not advisable, however, to place a refuse destructor in the midst of higher inhabited buildings, as the dust, with possibly an occasional escape of smoke, may cause some annoyance, though even this trouble can be obviated by proper attention to details in the design of the destructor building.

Types of Destructors.—In the forty plants inspected, ten different kinds of furnaces were represented, as follows:

Name of Furnace.	Number Inspected
Meldrum	13
Horsfall	10
Heenan	6
Beaman and Deas (Meldrum)	2
Warner	3
Fryer's Improved (Manlove-Alliott and Company)	2
Fryer's	1
Sterling	1
Baker's	1
Glasgow (local design)	1

All these destructors contain large brickwork chambers having fixed grates with boilers placed outside the refuse-burning portion. In

the destruction of refuse by fire, well-determined principles of combustion apply. In order that nuisance may be prevented, it is necessary that all combustible portions of the refuse shall be completely consumed, with the result of producing the highest state of oxidation. According to Professor Thurston, the requirements for success in burning wet fuel are:

"The surrounding of the mass so completely with heated surfaces and with burning fuel that it may be rapidly dried, and then so arranging the apparatus that thorough combustion may be secured, and that the rapidity of combustion may be precisely equal to and never exceed the rapidity of desiccation. Where this rapidity of combustion is exceeded, the dry portion is consumed completely, leaving an uncovered mass of fuel which refuses to take fire."*

In practice, the destruction of refuse may be attained successfully by burning it by forced draft in a so-called Dutch oven or chamber where the brickwork is maintained at a high heat, and the escaping gases are subjected to a high temperature with an excess of air for a sufficient length of time to oxidize the combustible constituents of the material. British destructors are designed in accordance with the above principles.

The forms or types of destructors vary, however, and for convenience may be divided into two general groups.

Group 1.—The first may be termed the mutual assistance type, where one unit contains several grates with divided ash-pits, the products of combustion intermingling in the upper portion of the furnace, thus combining several furnaces or cells in one. Representatives of this type are the Meldrum and Heenan.

Group 2.—The second comprises furnaces in which each burning grate or cell forms a separate unit. The products of combustion either commingle in a general flue or combustion chamber, or pass directly from cell to boiler. Representatives of the cell type in which the products of combustion intermingle in a common chamber before passing to the boiler are the Horsfall, Sterling, and Beaman and Deas (Meldrum). Representatives of the type in which the products of combustion pass directly from the cell into contact with the boiler are the original Fryer, Fryer's Improved (Manlove-Alliott and Company), Warner, and Baker.

The Meldrum, Heenan, and Horsfall types pre-heat the air used

* "Steam Boiler Economy," Kent.

for combustion to a temperature from 200 to 400° fahr. before it comes in contact with the burning fuel on the grate. Other makes of furnaces mentioned in Group 2 use air at ordinary atmospheric temperature. The utilization of heated air undoubtedly tends to more perfect combustion and higher temperatures both in cell and combustion chamber. Other differences in design, in the furnaces in Groups 1 and 2, may be noted, as, for instance, the drying hearth which some furnace makers consider essential in the destruction of refuse, the use of steam-jet blowers or fans for forced draft, the different provisions for arresting dust, the kind of boilers used, the various methods of feeding, clinkering, stoking, etc. All the above-named destructors, except the original Fryer, use forced draft, which is considered necessary for the attainment of a high temperature.

The aim in the design of refuse destructors should be to maintain a steady temperature. If it be considered that 1 250° fahr. is the minimum at which septic poisons in the products of combustion are destroyed, the higher limit of temperature is fixed by the materials used in the construction of the furnace. Temperatures greater than 2 000° fahr. are apt to result in high cost of repairs. Thus temperatures between 1 250 and 2 000° fahr. are desirable, both from sanitary and economical points of view. As the burning of refuse in a destructor is an intermittent process, requiring alternate charging and clinkering, the fluctuations in temperature should be minimized as much as possible. When destructors are of such design that the gases pass directly from cell to boiler without an intermediate combustion chamber, there is danger of unoxidized gases being cooled, by contact with the boiler, below the temperature required to prevent nuisance.

In general, modern British types of destructor vary in important details, and, of the different plants examined by the writer, those in which a combustion chamber or flue was placed between the cell (or unit) and the boiler, and where heated air was used for combustion, appeared to be doing the most satisfactory work.

Power from Refuse.—Of the forty plants inspected, all but two produced steam for power purposes, as follows:

Power Used For:	Number of Plants.
Electric lighting and power stations	16
Sewage pumping	7
Works purposes	14
Water pumping	1

In considering the utilization of power from refuse, it should be borne in mind that power is a secondary consideration, and that the primary object is to destroy refuse in a sanitary and economical manner.

At electric lighting and power stations, the demand for lighting purposes generally occurs for a short period in the evening, when an output very much higher than the ordinary working load is required. With refuse which is of low calorific value, and requires burning at a high rate to produce power, this means that the fires must be rushed, and consequently there is likelihood of incomplete combustion. Supplementary coal-fired boilers are usually found in connection with destructor-electric-lighting stations, or else the destructor is of much greater capacity than would be required to deal with the refuse only. The combination of refuse destructor and electric lighting plant may be economical, yet, if other means are available, whereby the power resulting from the destruction of refuse can be utilized regularly as produced and the furnace operated continuously at an easy working rate, this method should prove more desirable than electric lighting utilization. At the sewage pumping stations visited, the quantity to be pumped was usually insufficient to keep the destructor continuously in operation. Pumping water by the power produced from refuse would seem to be the most satisfactory method of utilization, but the conditions where such a system can be used are exceptional.

If no other outlet for the energy produced in burning the waste material is available, the power is used for what has been termed "Works Purposes," that is, crushing clinker, screening it into different sizes, mixing it with cement, or lime, to form bricks, slabs, etc., also for lighting the destructor depots or adjoining corporation yards, and the steam heating of neighboring public baths or libraries.

It is undoubtedly advisable to provide means for utilizing heat resulting from the destruction of city wastes, for the purpose of decreasing the cost of final disposition. At the same time, it should be kept in mind that a sanitary disposal of refuse is the primary consideration. There were indications at some refuse destructor plants in Great Britain of a tendency to slight the main factor of sanitary disposal.

Capital Cost of Destructor Installations.—The cost of construction of destructor installations varies greatly according to local conditions,

and figures for British plants would hardly apply in the United States. It appears from the data obtained, however (the details of which are given in Table 10), that the average cost of eighteen destructors would amount to about $4 470 per cell or grate, including the furnace with boiler and appurtenances, but excluding chimney, building and runway.

Cost of Operation.—The cost of operation, for different plants quoted in Table 10, was obtained from the engineer or superintendent in charge of the destructor or from the furnace makers. Some of the figures given are from official reports covering a year's work, while others are for short periods only. From the data, it appears that for twenty-four installations the average cost of labor per long ton (2 240 lb.) of refuse destroyed would amount to 24.3 cents, or 21.5 cents per short ton (2 000 lb.). As the American rate of laborers' wages is about double the British rate, this would make 43 cents per short ton of refuse destroyed on an American basis.

For supervision, only four installations had figures available, the average being 4.83 cents per long ton. Two plants reported the cost for repairs at 3.22 cents per long ton.

Only one complete report was obtained in which all charges for the destruction of refuse, including labor, supervision, interest on capital, sinking fund, repairs and supplies, were included. The total cost of operation, including all the above charges at Stoke-upon-Trent, amounted to $1.17 per long ton or $1.04 per short ton. By changing the labor rate so that it would compare with American conditions, and by assuming the same charges for interest, sinking fund and repairs, it would appear that the total cost of refuse destruction for a plant similar to that at Stoke-upon-Trent would amount to $1.50 per short ton in New York.

Of the various figures tabulated for the labor cost per long ton of refuse destroyed, it will be found that eleven destructors—in which refuse is fed into the furnace by hand—returned an average cost of 21.6 cents as against 27.3 cents for eleven top-fed destructors. Some figures used in making up the foregoing results for hand-fed plants covered only a short period of time. It is of interest, however, to note that hand-feeding does not seem to be more costly than top-feeding.

Refuse Burned per Man per Hour.—Definite information regarding the quantity of refuse handled per man per hour (assuming the

quality of labor to be comparable) affords a better general means for arriving at the labor cost of operating a destructor. From the figures for twenty-seven plants, on an average, each man employed would handle 0.78 long ton or 0.88 short ton per hour, varying from 0.5 to 2 long tons per hour with the type of plant and method of operation. At an easy rate of working, there should be no difficulty in destroying 0.75 short ton per man per hour; hence, with wages at 25 cents per hour (or $2 per day), the cost of labor would amount to 33⅓ cents per ton, while at 31¼ cents per hour (or $2.50 per day), the cost would be about 42 cents per ton.

Special Notes on Destructors.—In glancing over the various photographs which accompany this paper, it will be apparent that the English destructor is most substantially constructed, and that the buildings are intended for long service. From an examination of the various older furnaces, it would appear that the destructor portion, with ordinary care in operation, should last at least fifteen years.

As the appearance of a refuse installation has an important bearing on public opinion regarding the plant, it is of particular importance that the building should be made attractive, architecturally. The interior should have ample light, air, and provision for the comfort of the men employed, including baths and toilet facilities. These features have received consideration in Great Britain, as some of the photographs indicate.

Another factor requiring consideration is the extent of ground surrounding the destructor building. Ample land should be provided, if possible, so that dust may not cause complaint from neighboring householders.

Operation of Plant—Observations.—In the time spent at the various installations, many features of interest undoubtedly escaped attention, and perhaps some of the comments in Table 10 would hardly be warranted if some days instead of hours had been given to each. The recorded observations, however, indicate the conditions at the time the plant was visited. Generally speaking, from the appearance of the destructors, no special preparations were made because of the writer's visit, and in many cases no warning was given to those in charge.

Feeding—Charging.—Of the forty destructors, sixteen were of the top-fed variety, in which refuse was charged through ports on top of

the furnaces, and these sixteen represented nine different types. Three installations were top-fed by cart direct, and represented two types of furnace. By "cart fed direct" is meant that cart storage of refuse was necessary, and when the furnace was ready for charging each cartload was dumped directly into the cell. One furnace was top-fed by a so-called "tub-feeding" method. In this case, each cart was tipped at the ground level into a box or tub with hinged bottom doors. The tub was then elevated by a traveling crane, and the charge of refuse was dropped into the cell through water-sealed mechanically-operated doors on the top of the furnace. One destructor was top-fed by a patent charging truck. Here, also, refuse was elevated by power, dropped into a truck on wheels, and charged into the furnace as required. Eleven destructors were hand-fed at the front by shovel, and represented two types. Eight destructors were hand-fed by shovel at the back, and represented three types.

In commenting upon the different methods of charging destructors, it appears that the top-feeding method (except where water-sealed doors are used) allows smoke to escape. Even with water-sealed doors, smoke escapes when charging. Of the hand-firing methods, the front-fed type appears to be advantageous, with regard to concentration of labor and freedom from escaping smoke, but the storage bin cuts off light and air from the firemen, while some refuse may be mixed with clinker if the men are careless. With back hand-feeding by shovel, ample light and air can be given on the clinkering side of the furnace where it is most needed. As compared with front-feeding, back hand-feeding does not permit of the same concentration of labor, but allows greater comfort to the men employed, which more than compensates for this slight disadvantage.

In general, shovel-feeding obviates escaping smoke from top-feeding doors, allows a better selection of refuse, and does away with stoking to a great extent, as refuse can be charged directly on the grate, thus saving one operation in destroying the material. When the refuse has not reached an advanced stage of decomposition, and does not contain an excess of water, or such objectionable material as night-soil, hand-firing is undoubtedly to be preferred, especially for power plants.

Stoking.—By stoking is meant the dragging, pushing or spreading of refuse after it has been charged into the furnace. All top-fed

PLATE XLVI.
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REFUSE DESTRUCTION.

FIG. 1.—WATFORD DESTRUCTOR, SEWAGE PUMPING STATION, AND ELECTRIC LIGHTING PLANT.

FIG. 2.—WALTHAMSTOW DESTRUCTOR AT SEWAGE FARM, WITH PRECIPITATION TANKS IN FOREGROUND.

FIG. 3.—COMBINED ELECTRIC LIGHTING AND DESTRUCTOR STATION AT STOKE-UPON-TRENT.

FIG. 4.—COMBINED DESTRUCTOR AND SEWAGE PUMPING STATION AT WORTHING.

destructors and all destructors provided with drying hearths require considerable stoking. Hand-fed types, without drying hearths, where refuse is thrown directly on the grate, do not need much stoking.

At first sight, it would seem that refuse charged direct from cart to cell without intermediate handling should prove most sanitary and economical, yet the disadvantages of this method are many. For any particular case, a study of local conditions will determine the best system to be used.

Clinkering.—Clinkering is perhaps the most trying work in connection with the operation of a destructor. A mass of hot slag must be broken up by long bars, tipped into a wheel-barrow or other conveyance, and removed while in a highly heated condition. The work is performed by hand labor opposite the open doors of a highly heated furnace. There are various methods of conveying clinker, as by wheel-barrows, by cars on rails, or by skips running on an over-head rail. When cars or mono-railways are used, the storage room is limited, and the place where the material is deposited must be cleared at intervals; for this reason, the system has been abandoned in favor of wheel-barrows at many plants.

Various mechanical devices, such as tipping grates, etc., have been tried in order to lessen the work of clinkering, but, up to the present time, all have failed. At Westmount, in Canada, the destructor site was well adapted for a clinker pit, whereby clinker, instead of being removed in barrows or cars while hot, is dropped through a trap-door in front of the furnace to a lower level, where it is allowed to cool. This is a decided advance over the usual practice. A further improvement might be made at Westmount by enclosing the pit and utilizing the heat contained in the hot clinker for raising the temperature of air for combustion.

Character of Clinker.—As a general rule, the clinker in sight at the various installations was found to be hard and well-burned, except where plants were carelessly operated, or where fires were rushed at some electric lighting stations. In order that clinker shall be dense and that practically all the carbon shall be oxidized, it is necessary that the clinker be exposed to a high temperature for a sufficient time to consume thoroughly all the combustible material.

General Notes on Operation.—As with other works, the method of operating a destructor may mean its success or failure. A well-de-

signed plant poorly operated may give rise to nuisance, whereas a poorly-designed plant efficiently operated may cause no trouble whatever. At most installations it was manifest whether or not the authorities were interested in the sanitary disposal of refuse. In poorly-operated plants, dirt, dust, and smoke were in evidence, while in efficiently-managed installations, cleanliness, order, and system were the rule. Of all destructors visited, there were few in which dust, either from the refuse on storage, or from the process of clinkering, was not a cause of inconvenience, though the trouble was usually confined to the destructor building.

The necessity for a systematic routine in the operation of refuse destructors was quite apparent, and in some of the plants the firemen worked practically by the clock. Each step in the process of feeding, stoking, or clinkering was performed regularly at stated intervals, thus tending to efficiency in management. Few self-recording devices or checks on the operations, such as steam gauges, chimney gas analysis apparatus, draft gauges, pyrometers or thermometers, were noticed at the various plants.

Temperatures.—At each destructor in operation a sight estimate of the temperature of the main flue or combustion chamber was noted, as this factor has a decided bearing upon the freedom from nuisance and the efficiency of any installation. It is of great importance, both from sanitary and economical points of view, that temperatures be regulated between a lower limit of 1 250° fahr. and a higher limit of perhaps 2 000° fahr. When the temperature is high, the gas escaping from the chimney is almost colorless, but when charging or clinkering operations are in progress, murky to dense white smoke may be apparent for a short time. As a rule, gases escaping from the destructor chimneys showed no color when compared with the black clouds emitted from the chimneys of neighboring manufacturing establishments in Great Britain.

Utilization of By-Products.—By-products resulting from the destruction of refuse, excluding the steam generated by the heated gases, consist of clinker from the grates, fine ash from the ash-pit, and flue dust from the combustion chamber or flues, besides tins, bottles and earthenware which may or may not be passed through the furnace.

Clinker.—Clinker, when burned to a hard vitreous mass, is utilized as an aggregate, mixed with cement, and made into slabs, bricks or

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FIG. 1.—SWANSEA DESTRUCTOR AND ELECTRIC POWER HOUSE.

FIG. 3.—HAMMERTON STREET DESTRUCTOR WORKS, BRADFORD.

FIG. 2.—COMBINED DESTRUCTOR AND ELECTRIC LIGHTING STATION AT WESTMOUNT, CANADA.

FIG. 4.—BATLEY COMBINED DESTRUCTOR AND ELECTRIC POWER STATION.

mortar, usually by hydraulic presses or mortar mills. If not thus utilized, it may be used for road bottoming, sidewalks, or for filling low land. The question of clinker utilization is usually decided by local conditions. Where clinker can be sold or used advantageously in making slabs, bricks, etc., machinery is usually installed, and the process results in reducing the total cost of destruction. At Worthing, on the south coast of England, clinker has been mixed with tar and used as a road pavement. The revenue derived from the sale of clinker will vary with the demand in any locality. Six municipalities report clinker sold at an average price of 42 cents per long ton.

Flue Dust.—In burning mixed refuse, some fine incombustible material finds its way into the destructor flue or dust traps, and must be removed periodically. This may have an important bearing upon the actual capacity of a destructor, as it may be necessary to shut down the plant for several days while the cleaning process is under way. The time elapsing between cleaning periods varies with the character of the material destroyed and the type of destructor. It would appear to be necessary to clean out all flues thoroughly once every 4 to 6 weeks in Great Britain, except where a special form of dust-catcher is used.

Flue dust has been used as a base for disinfecting-powder.

Tins.—Refuse contains a large quantity of tinware, varying in size from food cans to boilers and wash-tubs. These articles are not usually put through the destructor, but are thrown to one side and sold in bulk or compressed by a machine into suitable bundles. At some plants, solder is melted from tins in a special furnace, and the resulting products, solder and iron, are sold to junk dealers. The revenue from the sale of tins will depend on local considerations.

Power from Refuse.—Figures for eighteen destructor tests, giving the quantity of water evaporated per pound of refuse ("from and at 212° fahr.") for periods varying from 6½ hours to one year were secured. The highest rate of evaporation was 2.66 lb. of water per pound of refuse, in a 15-hour run at a destructor in a colliery district. The lowest gave 0.88 lb. of water per pound of material, in a test of 11½ days, with refuse containing a large proportion of night-soil. The average evaporation in eighteen modern destructor tests amounted to 1.62 lb. of water per pound of refuse. In all the foregoing figures the water evaporated is a gross amount, and in order to obtain the

net useful steam produced for power purposes it is necessary to deduct for forced draft apparatus. It appears, from the figures quoted, that, in a district where coal is abundant and cheap, it is possible to evaporate about 2.5 lb. of water per pound of refuse, while in other districts, distant from coal fields, destructors are capable of producing an evaporation of about 1.5 lb. per pound of refuse.

A test of a refuse destructor for a few hours hardly gives a safe figure upon which to base conclusions on power production, as the material may vary in character and calorific value with the season of the year. It is of interest to note in Table 10 that the destructor at Westmount, Canada, evaporated 1.36 lb. of water per pound of refuse in a test run of 8½ hours on May 2d, 1906. The refuse on storage at Westmount in August, 1906, was much drier, and contained a larger proportion of rubbish and ashes than the material for a similar period in Richmond Borough.

Deductions.

From data and observations made upon the various destructors, the following deductions were made:

Nuisances or Possible Cause of Complaint.—This heading means that, of the plants examined, some might, if situated in a critical position, give rise to complaints by inhabitants of the neighborhood. There is no intention of condemning any installations on the score of nuisance, as no doubt any destructor which had proved objectionable would have been closed by legal procedure.

It is possible that, of the forty installations, nine might cause complaint through the escape of unconsumed gases, poor operation of plant, or bad design.

Objectionable Features.—At the forty destructors an objectionable feature common to all was that due to the escape of dust, either from the refuse on storage, from the clinkering operations, or during the removal of flue dust. This dust, unless it escapes through the chimney, becomes a nuisance which affects only the workers about the plant, and anything that may be done to minimize it will mean a decided advance over present practice.

At sixteen plants, smoke or unconsumed gases were escaping through feeding ports, stoking or clinkering doors.

The system of storing refuse at six installations was objectionable because the firemen were obliged to work in the material when charg-

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FIG. 1.—CLINKER CAR, BROMLEY.

FIG. 2.—CLINKERING FLOOR AND DESTRUCTOR UNITS AT KINGS NORTON.

FIG. 3.—CLINKER YARD AT KETTERING, WITH HOUSES ADJOINING DESTRUCTOR BUILDING.

FIG. 4.—CLINKER RAILWAY AND YARD AT SUNBRIDGE ROAD DESTRUCTOR, BRADFORD.

ing. At six plants the working space was cramped, and there was a lack of light and air, causing discomfort to the men. At two destructors the method of feeding refuse was objectionable because of complicated mechanical devices. At only two out of the forty plants were unconsumed particles of organic matter noted in the clinker. This was undoubtedly due to lack of efficient operation, or negligence on the part of the firemen.

Commendable Features.—Summing up the commendable features of the different installations, it may be said that practically every plant had some feature which would call for approval. In general, all the destructors were well constructed and designed for hard service. The buildings were usually of brick, substantial, and in some cases of attractive appearance. With but one exception, all the destructors were disposing of refuse without the use of additional fuel, and in the case noted, coal was used only during wet periods in summer.

Temperatures were generally sufficiently high to prevent the escape of unconsumed gases. Steam raising was practiced at all but two of the plants, and the amount of power produced had an important bearing on the economy of the process. Clinker, resulting from the burning of refuse, was sold or utilized for different purposes, tending to further economy in operation, while flue dust and tins were utilized.

Mixed-refuse destruction, whereby waste material discarded by householders is disposed of without nuisance, and the resulting by-products are turned to useful purposes, would seem to be an ideal system. Certainly, in so far as sanitary disposal is concerned, mixed-refuse destruction, efficiently conducted, should cause no trouble if the material be of fair quality. While it is not possible at the present time to do more than estimate the cost of such a method in the vicinity of New York, it would appear that mixed-refuse destruction should not cost much more than garbage cremation in Richmond Borough, while the sanitary advantages accruing would more than compensate for any slight excess in cost.

Recommendations for a Mixed-Refuse Destructor Installation in the Borough of Richmond, West New Brighton District.

In view of the local experiments and experience gained abroad, the details of which are partially recorded herein, the writer recommended for the first installation at West New Brighton, in the Borough of Richmond:

1.—A hand-fed destructor charged at the back of the furnace and clinkering on the opposite side or front of the furnace.

2.—That refuse be stored in a bin or hopper with a door or curtain to control and prevent the escape of dust into the destructor room while the hopper is being filled.

3.—That refuse be dumped into the bin or hopper behind closed doors; and that the refuse storage room be separated from the destructor portion of the building.

4.—That heated air be required for the combustion of refuse.

5.—That a water-tube boiler be specified.

6.—That steam-jet blowers, or fan-draft, or both, be provided so that the advantage of either may be determined.

7.—That the air for forced draft be drawn from the upper portion of the tipping-room and feeding or clinkering-room, so that positive ventilation may be secured.

8.—That the clinkering process be arranged so that hot clinker is dropped into a pit and the heat from the clinker is utilized in raising the temperature of the air for combustion.

9.—That ample working space, light, and air be provided in the building, and the plant be located so as to cause no trouble from escaping dust.

10.—That a suitable mess-room, bath and toilet-room be provided, for the comfort of the men employed.

11.—That the exterior of the plant be made attractive in appearance.

Contracts have been entered into with Messrs. Heenan and Froude, Ltd., for the erection of a 60-ton destructor, and with McHarg-Barton Company for the erection of the building, chimney, etc., at West New Brighton. The plant will probably be in working order before the close of 1907, and later, the writer hopes to present and compare the actual results with the preliminary studies herein outlined.

In conclusion, the writer desires to acknowledge his indebtedness to the Hon. George Cromwell, President of the Borough of Richmond, for his interest and encouragement; to Louis L. Tribus, M. Am. Soc. C. E., Consulting Engineer to the Borough President, for advice and support; to B. F. Welton, Assoc. M. Am. Soc. C. E., for his interest in the calorific tests, and for many valuable suggestions; to George Wood, Assoc. M. Am. Soc. C. E., for assistance on different portions

PLATE XLIX.
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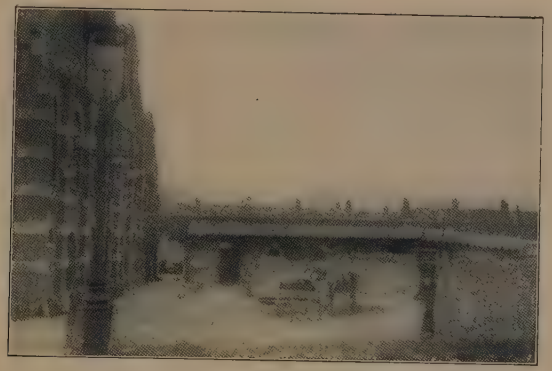


FIG. 1.—WOOLWICH DESTRUCTOR YARD. PILES OF CLINKER BRICK.

FIG. 2.—MORTAR MILLS AT SUNBRIDGE ROAD PLANT, BRADFORD.

FIG. 3.—YARD AT WALTHAMSTOW, WITH PILES OF SLABS MADE FROM CLINKER.

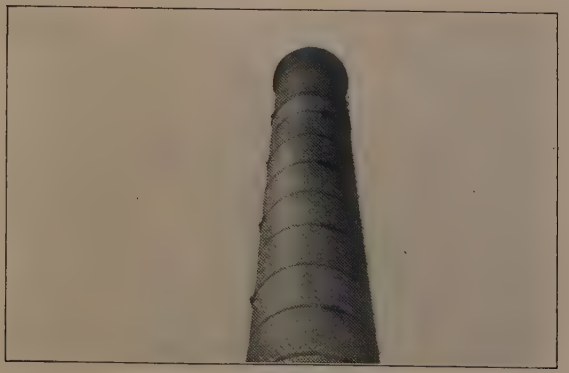


FIG. 4.—CHIMNEY AT WESTMOUNT, WITH DESTRUCTOR IN OPERATION.

of the work; to the representatives of the different firms engaged in destructor building, particularly Messrs. W. F. Goodrich, Frank Rudder, and George Watson, and also to the various municipal engineers, surveyors, superintendents, and managers of the British destructors examined for their courtesy and assistance.

DISCUSSION.

Mr. Venable. W. M. VENABLE, M. AM. SOC. C. E. (by letter).—In this valuable paper the author proves that the refuse of the Borough of Richmond, New York City, contains sufficient calorific material to enable it to be burned without offense, and without using auxiliary fuel. He also presents data regarding forty incinerating plants in Great Britain, with the object of determining the best features of design for use in a proposed plant. The investigation which led to the conclusion that it is desirable to burn all the refuse in one incinerator, if that is found practicable, is not given in the paper, nor is there any investigation of the merits or demerits of incinerators of American design. The writer is of the opinion that, in the United States, it is seldom desirable to reburn all refuse, including the ashes from private houses and other buildings, and would like the author to present in detail the data upon which this determination, which preceded the investigation reported in the paper, was based.

Whether or not the method of destroying all wastes in one set of furnaces will be found the best for municipalities generally, engineers are indebted to Mr. Fetherston for his thorough work in ascertaining the quantities of garbage, ashes and rubbish, and their calorific value, in what may be taken as a representative district. It is remarkable, also, that the best summary of British practice in refuse disposal is found in this paper, by an American engineer, for use in America. Too much praise can hardly be given for the judgment shown in the preparation of the various tables, although Table 1 was prepared so long ago as to make it necessary for the reader to guard himself against the error of assuming that it contains all the data now available on the subject with which it deals. The work of Messrs. H. de B. Parsons, Rudolph Hering, W. F. Morse, and others has been published since 1904.

From this paper and other available data, it is safe to assume that, in almost any municipality, if all the household refuse is collected and brought to one place, the mixture will contain sufficient calorific energy to make it practicable to burn it without admixture of other fuel, and to permit the generation of some steam for power purposes from the heat in the gases of combustion. It does not follow from this, however, that such collection and disposal is the most advisable. It can hardly be granted, as a general proposition in cities, that it is impossible to collect ashes in such condition that sanitary disposal of them without reburning is impracticable. If such is granted, as appears to have been done in the present case, there would still be reasons for considering separate collection and burning in separate parts of an incinerator, keeping ashes separate from garbage and refuse, both for sanitary reasons and for convenience and economy in actual burning.

Mr. Venable.

If the reburning of ashes is to be decided from considerations of economy only, it should be regarded entirely apart from the disposal of other wastes, for the introduction of ashes into the garbage makes their disposal much more costly than otherwise, even if it is necessary to furnish a considerable quantity of coal to assist in destroying the garbage.

While ashes from household fires contain much combustible material, they do not contain enough, as a rule, to make up for the cost of stoking them through a crematory, not including plant charges; and, unless a very great reduction in weight is secured by reburning, there will be no saving in total haul by the burning process. Generally, the weight of ashes passed through a crematory is not very greatly reduced, although the weight of rubbish and garbage is very much decreased by burning. There may be cases, however, where a furnace can be located at the center of a district, and the haul to the dump is very much longer than that to the furnace, in which cases the saving in haul will more than counterbalance the cost of dumping, stoking, interest and depreciation on plant, and reloading for haul to the dump.*

In the United States it has been customary to dispose of ashes separately from garbage, from motives of economy, and furnaces for the disposal of garbage or refuse, or of both combined, have been designed with the expectation that ashes would be excluded. It is practicable to burn these materials properly without forced draft, and several builders of crematories have accomplished this successfully, at prices of disposal per ton quite as low as those obtaining in England for the mixed refuse; but crematories operating on natural draft can be abused more readily than those using forced draft, and, consequently, when handled by the ignorant persons who are so often placed in charge, the furnaces have received the blame that ought to have been charged against the persons in authority. Of course, very many crematories of poor design, and crematories attempting to burn materials for which they have not been fitted, have been installed, and the blame has not always been with the operator. Crematories of the so-called American design are much cheaper to build than those of the British type, as they require no boiler plant, or power auxiliaries. They will consume successfully garbage and rubbish of a character which cannot be burned in those of the British type, and are very economical in the use of labor in stoking. Therefore they ought not to be condemned, or left out of consideration in selecting a method of disposal, but should be installed where it is shown that they will be most economical in the long run; and proper precautions should be taken that they are operated so as not to produce a nuisance.

* This matter is discussed in the writer's book, "Garbage Crematories in America," in which will also be found descriptions of every type of crematory installed in the United States, reference to every United States patent of interest in this field, and a list of the more important and representative plants installed by each builder of such works in the United States.

Mr. Venable. On the other hand, it is practically impossible to burn ash-bin refuse with natural draft. The reason is, not that a strong enough natural draft cannot be obtained, but that the constant opening of doors for stoking, on account of the large proportion of ashes to actual fuel in the mixed refuse, admits to the furnace too much air for proper combustion. This reduces the draft and also causes the production of foul odors in the chimney gases. It requires so much more head to create a proper draft through a mixture of ashes than through a mixture of rubbish that it is possible to burn the rubbish without offense, on natural draft, even with doors frequently open, although it is not possible if a large proportion of ashes is introduced into the mixture to be burned. Thus, practically all British incinerator builders have been compelled to adopt forced draft because they reburn ashes, and to install boilers in order to develop power to obtain it. When forced draft is used, the stack should be designed merely to carry off the gases of combustion, not to produce any portion of the head across the grates. Thus, when the stoking doors are opened, there is no tendency to draw the air in the stoking room into the furnace; but, on the other hand, there may be a tendency for the heated gases within to come out through the open doors, as observed by Mr. Fetherston in several British installations. Forced draft, subject to close regulation, is preferable in any installation, and is a very great safeguard against the admission of too much air into the furnace, above the fires; but it is not the only way in which this can be safeguarded, and, in many plants, especially in the smaller towns, the advantages to be derived from the installation of a boiler are not as great as the disadvantages.

These remarks may be considered as not properly applying as a discussion of Mr. Fetherston's valuable paper, one of the premises of which is that the ashes are to be burned. While fully recognizing this, as a condition precedent to his inquiry, and having no quarrel with it, in the case of the Borough of Richmond, the writer has ventured these remarks as perhaps of some interest to others, for conditions which may differ from those stated in this paper.

Mr. Cary. ALBERT A. CARY, ESQ.—This paper, viewed from the standpoint of a furnace and fuel specialist, is of great interest to the speaker, who, having had considerable experience in burning various fuels of low calorific value and also fuels carrying large percentages of moisture, such as spent tan bark, wet refuse wood-pulp shavings, spent licorice root, bagasse, etc., can well appreciate the difficulties encountered in burning wet municipal wastes; and burning them so as to obtain sufficient heat for steam-making, which heat is in excess of that required to evaporate the moisture contained in the fuel, and to dissociate the fuel (thereby liberating the volatile gases it contains, which action is necessary before these fuel constituents can burn).

Mr. Cary.

If any fuel be dried and an analysis be made of its chemical composition, then, by the use of a modification of the well-known Dulong formula, a determination of the calorific value of the sample analyzed can be made. This may or may not be of use in furnace determinations, depending on the nature of the fuel and the value of the sample as a fair representative of the entire mass of fuel consumed.

It is no easy matter to obtain a representative sample of the entire fuel consumed during a test, even when the fuel is fairly uniform in quality, but when its quality is of a very variable nature, such as in refuse-burning plants, the difficulties in obtaining a small sample of fair average value become almost insurmountable. Aside from this difficulty, after making calculations from the chemical analysis of a dried sample, one does not obtain a true calorific value of the fuel, as this process of determination assumes that the combustion is wholly an exothermic process, that is, one producing heat with no heat absorption occurring for internal or external reactions. Such endothermic actions always take place in the process of combustion, as it requires heat energy to break up solid masses of fuel and liberate and split up the hydro-carbons, to say nothing of the energy required to evaporate the moisture, both on the surface and contained hydroscopically.

For this reason, when careful tests are made, the calculated fuel values are found to be higher than those obtained by using the oxygen fuel calorimeter; the difference between these two values indicates the heat energy absorbed by endothermic reactions.

Coming now to the fuel calorimeter, reference will be made only to the work done in the Mahler bomb. By a proper manipulation of this apparatus, there is no difficulty in determining the true value of the sample tested, and the results obtained will require no corrections for chemical endothermic actions; but here, also, there is difficulty in obtaining correct samples, representing a fair average of the whole fuel mass, and it must be remembered that the quantity of fuel tested weighs only 1 g. (that is, less than 0.04 lb.).

The great difficulty in obtaining the true calorific values of the refuse by either of these methods, therefore, can be well appreciated, and the question naturally is: How can this most important value be determined?

The answer is prompted by a somewhat extended experience in making furnace investigations leading to accurate heat balances. Heat balances are usually obtained by calculations made from the analysis of the fuel, the quantity of fuel used, the analysis of the products of combustion, and a proper consideration of the various furnace losses.

By a somewhat reversed method of calculation, made from a series of observations, the chemical composition of the fuel may be obtained,

Mr. Cary. and from it the unknown quantity to be determined; then, with a very fair degree of accuracy, its value may be found.

The accuracy of such a determination is, unquestionably, far greater than may be obtained by any system of sampling when such a mixed fuel as municipal wastes is used.

This method was used in the work of Mr. C. E. Stromeyer, referred to on page 369, but he did not carry his work far enough to obtain sufficiently satisfactory results.

The furnace gas analysis becomes a most important matter in such test work, and the mere finding of the percentages of CO_2, O, CO and N, by difference, by the use of the ordinary Orsat apparatus, will not give sufficient information, as experience has taught the speaker that in such work it is necessary to determine the free hydrogen and hydro-carbons as well.

Mr. Stromeyer also relates, in his report, the failure of his high-temperature measuring apparatus, which furnishes most important information. The speaker is continually using such apparatus, without trouble, in furnace tests where much higher temperatures exist.

The speaker does not wish to be understood as criticizing Mr. Stromeyer in this work; on the contrary, he regards it as very much in advance of any testing work previously done in refuse crematories. He merely wishes to indicate that this is the most reliable way of obtaining this much-sought-for information, when proper testing is done.

To obtain data needed to make such fuel determinations, one does not require any other apparatus than that used in making complete and exhaustive furnace tests, but careful refinements must not be neglected, both in applying and using the apparatus and in having them all carefully calibrated.

The work done by Mr. Fetherston, as shown in this paper, to obtain such information, by fuel sampling methods, is certainly highly creditable, and the amount of work involved appeals to the speaker strongly, as he knows by experience what it means.

Concerning the large percentages of moisture held in fuels, the speaker has profitably passed very wet fuel between a pair of large cast-iron rolls, with rough faces, one roll being of a little greater diameter than the other. These rolls, both running at the same number of revolutions per minute, were held together by large springs which allowed them to separate when solid chunks reached them.

In this way a large quantity of the contained moisture was squeezed out, and higher temperatures were obtained in the furnace, as well as better combustion, for the furnace is the poorest place in the world to evaporate water.

Mr. Fetherston's statement of the requirements necessary for burning wet fuel, on page 377 which, as he states, originated with Pro-

fessor Thurston, may be found in the *Journal* of the Franklin Institute for 1874, where it will be found to refer especially to spent tan and wet saw-dust. Mr. Cary.

For the combustion of moist fuel, the highest furnace temperatures possible are most essential, and that requirement is one of the weakest features in general garbage incinerating plants. It is firmly believed that much profitable development is possible in this direction.

The disposition of the highly heated surrounding surfaces mentioned is a matter of much importance with such fuel, and combustion chambers must be proportioned to the amount of gaseous matter and moisture given off by the fuel.

The speaker can hardly admit the statement indicating that combustion should be retarded and limited by spots of dry fuel forming on the grate and burning to expose wet fuel, thereby stopping combustion. Such conditions should never exist, as they indicate bad design.

To obtain the most desirable results, the combustion of the fuel should constantly be accelerated.

Pre-heated air, introduced under some pressure, to offset its dilated condition, will assist in producing such results, as is noted by Mr. Fetherston.

Steam jets should certainly be avoided as much as possible, as there is altogether too much steam given off from the fuel in the furnace, and steam has a cooling effect on the fire-bed. To obtain the best results, the steam used to disintegrate the clinker should be superheated.

The speaker cannot agree with Mr. Fetherston when he places the minimum desirable furnace temperature as low as 1 250° fahr., which is dangerously near the lowest temperature at which some of the gases found in the furnace will ignite. Such a temperature will surely be followed by most imperfect combustion.

A furnace should not fall below 1 800° fahr., as experience proves that, under lower temperatures, both furnace and boiler efficiencies drop. Further, 2 000° fahr. is too low for a maximum temperature, as the speaker's best furnace results have always been obtained with temperatures of 2 500° fahr., or greater.

If a furnace is properly designed and built, there is no reason why it should not be durable under a temperature of 2 500° fahr., and with destructor furnace conditions.

The speaker's experience, of many years in furnace work, has taught him that proper provision for great expansion and contraction is frequently neglected, and, also, that high-grade refractory materials are not used as much as they should be, and that high-grade furnace masons are not employed, but, where all these requirements are met, the durability of furnaces is greatly increased.

On page 375, it is noted that Mr. Fetherston assumes a combined

Mr. Cary. furnace and boiler efficiency of 50 per cent. By the system of testing referred to in this discussion, the exact efficiency of the furnace can be obtained. The information thus obtained will also point out definitely the exact causes of inefficiency, and thereby lead to a rapid, rational, and scientific development and improvement of the system of garbage incineration; and the time is certainly favorable for work of this nature, as shown by Mr. Fetherston's earnest work and careful investigation of existing conditions.

Mr. Foster. E. H. Foster, M. Am. Soc. C. E. (by letter).—The valuable data which Mr. Fetherston has presented in this paper will certainly be appreciated by engineers who have occasion to explore this comparatively obscure field, and it is certain that the paper will prove an important addition to the Society's *Transactions*.

Attention is called to the quotation on page 377 from Professor Thurston, giving the requirements for success in burning wet fuel: To insure that "the rapidity of combustion may be precisely equal to and never exceed the rapidity of desiccation" offers a condition which is to be steadfastly striven for, but which, unfortunately, can only be obtained under the most ideal conditions, and one can only hope to fulfil it when the combined collection of the city's waste is to be burned. So long as furnaces are required to burn garbage only, or garbage and rubbish, special provision must be made for carrying out the above requirements. When garbage alone is burned, fuel must be added to support combustion. Professor Thurston's remarks show why coal should be used, and not oil or natural gas, since it is the heated mass in the coal, and not the volatile matter, which accomplishes the drying process. The quality of coal need not be high; in fact, coals of the poorer quality, containing high percentages of ash, are really more suitable for this purpose. When garbage or rubbish have to be dealt with, without ashes, some other means must be adopted for preparing the garbage for burning, and, whatever method is used, it must be carried out inside the furnace, thus it becomes a part of the detail of the design.

It must not be considered that the chief desideratum is to obtain the highest possible temperature in any part of the furnace. Such an impression would be entirely wrong. The temperature, on the contrary, must be maintained between certain moderate limits, preferably between 1 800 and 2 000° fahr., but with a minimum never less than 1 250° fahr. The disadvantages of too high a temperature may be stated as:

Excessive cost of repairs;

Melting of the dust and clinker, causing it to stick to the fire-brick linings inside, and loss of time and labor in cutting out, periodically, by hammer and chisel, the slag-like accumulations;

Discomfort to the operators in removing the clinker from the furnace.

Mr. Foster.

The limit of low temperature is reached at the point where the gases of combustion cease to be dissociated and oxidized. It is necessary, then, to maintain a temperature well above this, thus rendering them thoroughly innocuous, and without which no process of destruction may be termed sanitary or without nuisance.

In the study of designs of various furnaces intended for destroying refuse, attention has been drawn to the conservation of the heat by various recuperating devices, such as an air heater for extracting the heat from the flue gases and transferring it to the air which is being fed to the furnace, and the method of cooling the clinkers by taking up their heat in the same air going to the furnaces. All these devices, which resemble the conventional economizers and air heaters used in connection with steam-power plants for securing higher efficiency and economy in fuel consumption, serve an entirely different purpose in the case of destructor furnaces. They are rendered necessary in order to insure a high minimum temperature which must at all times be maintained, the character of the material fed to the grates being of such a nature that these precautions are necessary.

Whereas the steam generated from the plant represents a valuable asset, and, in some instances, can be made to do useful work, doubtless there will be many cases where, in the absence of a suitable method of utilizing this power, the steam must be blown off and wasted. As it is the condition in the furnace which is of most importance, even if the steam generated is wasted, the devices for recovering the heat must be used.

On page 383 Mr. Fetherston suggests a further improvement which might be made in the Westmount plant, namely, to utilize the heat contained in the hot clinker for raising the temperature of the air for combustion. This idea is being carried out in the city plant now under construction.

It is a mistake to rely on the recommendation that the conversion of the power of the destructor plant into electrical energy is the most suitable outlet for that power. Mr. Fetherston mentions the pumping of water or sewage as an appropriate use. A still more appropriate use would seem to be the manufacture of ice, for which such a plant is strikingly well adapted. For instance, with an absorption ice machine, 9 lb. of ice may be readily procured by the burning of 1 lb. of coal under the grates of the boiler, whereas, in a destructor plant of 50 tons capacity, 1 lb. of steam may be readily evaporated per pound of mixed refuse destroyed. A 50-ton destructor plant would serve a community with a population of approximately 40 000. By comparing these figures it will be seen that a 50-ton refuse destructor will produce 50 tons of ice per day, or an allowance of 2½ lb. of ice per capita, which would be a liberal amount.

Mr. Foster. An important feature in the design of furnaces is the avoidance of smoke; this can only be accomplished by isolating completely the furnace and combustion chamber from the water-heating surface connected with the boiler, as the chilling effect produced by contact of the partially cooled consumed gases against the cold surface of the tube containing water will suppress complete combustion and result in a smoky chimney.

Mr. Welton. B. F. Welton, Assoc. M. Am. Soc. C. E.—The speaker has followed with great interest the development of the work done by Mr. Fetherston, the results of which are so admirably presented in his valuable paper.

The speaker's relation to this work, as stated by the author, has been in connection with the determination of the calorific values incorporated in the text of the paper. The purpose of this discussion, therefore, is to describe in some detail the methods used in making the calorific tests and proximate analyses, in order that the reader may be enabled to form his own estimate of the relative value of those results as compared with similar tests of other and more homogeneous materials used as fuel.

It is also desired to record the results of a series of chemical analyses of the component parts of the refuse made in the same laboratories by Professor Stephen F. Peckham, Member of the American Chemical Society, whose assistance has been highly appreciated.

The primary purpose of the experiments was to provide fundamental data from which could be determined the feasibility of the sanitary disposal of the wastes of the Borough of Richmond by self-combustion, in a refuse destructor of the same general type as used in Great Britain.

Inasmuch as the matter of heat utilization and power production was to be taken up ultimately, in connection with the disposal of the refuse, the results of the experiments were also to be considered as possibly affecting the design of the destructor. If the results of the calorific tests should show that the material was not suitable for self-incineration, it was hoped that the chemical analyses might provide the necessary information for determining some alternative method of sanitary disposal. On the other hand, if the material should be found suitable, the chemical analyses might furnish additional data for the study of means for the prevention of possible nuisance by the escape of the products of combustion, or for the recovery of commercially valuable material.

After the conclusion of trial calorific tests on two sets of samples to ascertain what methods of handling the material in the laboratory would secure the desired uniformity of results, a consultation was held between the author and the speaker to define the scope of the experiments.

It was decided: Mr. Welton.

First.—That, if the experiments were to be conclusive, they should be extended over a period of at least a full year, thus showing the entire seasonal variation in the character of the collections, which variation, it was thought, might be sufficient to interfere, perhaps, to a serious degree, with the successful operation of a destructor;

Second.—That the samples should be taken with sufficient frequency and in such manner as to be truly representative of the collections, both as regards the character of the material and the period covered.

It was finally settled that the samples submitted to the laboratory should represent the daily collections of the Bureau of Street Cleaning for a period of about two weeks, or a half month.

The primary sampling from the actual collections, as well as the initial preparation of the half-monthly samples, was to be made under the direction of Mr. Fetherston.

The sampling, as described in detail by the author, consisted in the selection of representative material which was subsequently separated, by sieves and hand-picking, into six general classes as follows:

1.—Garbage,
2.—Coal and cinders,
3.—Rubbish,
4.—Fine ash,
5.—Clinker,
6.—Incombustible material.

The garbage consisted of vegetable and animal matter, etc., such as ordinarily collected from dwellings.

The coal and cinders was the better portion of the stove and furnace wastes of the district.

The rubbish consisted of a variety of materials, such as paper, excelsior, rags, fibrous material, etc.

The fine ash was the material from the general collections which would pass through a screen of $\frac{3}{8}$-in. mesh, and consisted principally of the finer residue from domestic fires.

The clinker was that contained in the residue from domestic fires, and those of schools, churches, etc.

The incombustible material was largely glass, metal, stone, bricks, etc.

The initial preparation of the samples comprised the reduction of large quantities of material of the several classes by quartering, the evaporation of nearly all the moisture, and the rough pulverizing of all samples to effect a uniformity which would serve to make the samples submitted to the laboratory truly representative.

Mr. Welton. The weight of these samples was approximately:

1 lb. of garbage (dry);
2 lb. each of coal and cinders, clinker and fine ash;
¼ lb. of rubbish.

The condition of the samples, as they arrived at the laboratory, after going through this preliminary process, was about as follows: The garbage, in the majority of cases, was fairly dry, but soft and greasy; most of it would pass a sieve of ½-in. mesh, and, while the odor was decidedly in evidence it was not offensively so. Nearly all the coal and cinders would pass a sieve of ⅛-in. mesh, and showed a large proportion of unburned coal.

The fine ash and clinker were in about the same condition as the coal and cinders, except that the difference in the quantity of carbon present was plainly evident from the color and general appearance.

The rubbish presented the appearance of shredded rags, paper, etc. No incombustible material was tested, for obvious reasons.

Upon arrival at the laboratory, each sample was immediately placed in a wide-mouthed glass jar with a ground-glass stopper, and as soon as convenient thereafter a careful determination was made of the contained amount of moisture. This operation was conducted using about 10 g. of garbage and about 5 g. each of the other samples.

The whole of each sample of garbage and rubbish was then made to pass a sieve of No. 20 mesh by repeated grinding in a small pulverizer of the coffee-mill type. The coal and cinders sample was pulverized in a laboratory ball mill until it would all pass a No. 40 sieve. The clinker and fine ash were treated in the same manner.

The samples were then replaced in their respective glass jars and thoroughly mixed by agitation.

Proximate analyses were next made, determining again the moisture, and, in addition, the volatile matter, fixed carbon, and ash. For these determinations, the following weights of material were used:

Garbage	about 1.5 g.
Coal and cinders	" 2.0 "
Clinker	" 2.0 "
Fine ash	" 2.0 "
Rubbish	" 0.5 "

These quantities of the several materials were taken at random directly from the jar containing the whole sample, since it was found that practical duplication of results could readily be obtained without further reduction in size or quantity of the sample.

All determinations of moisture were made by using an electric oven kept at a constant temperature of about 180° fahr. The coarse samples were allowed to remain in the oven for about 18 hours, but

only about 1½ hours were necessary when the samples were in the pulverized condition. The volatile matter was determined by placing the dried material in a small, covered, porcelain crucible, over a three-flame Bunsen burner, care being taken that all the carbon deposited during the combustion of the volatile hydro-carbon was afterward consumed by the Bunsen flame. (Platinum crucibles were first used for this work, but some constituent of the coal and cinders, which was later discovered to be tin, probably from tin cans, re-acted with the platinum, ruining the crucibles, and their use was abandoned). Mr. Welton.

The fixed carbon then remaining in the crucible was next reduced to ash by open burning over the same Bunsen flame until no loss in weight occurred.

The percentages of the various determinations were reduced by calculation to the basis of the condition of samples as they were received at the laboratory, and the garbage analyses were still further modified to represent the conditions in the original sample before evaporation of any of its moisture, a record of the evaporative tests being sent to the laboratory with each sample.

The calorific values were determined by the Mahler bomb calorimeter, which provides for the combustion of the material in the presence of oxygen at a pressure of 25 atmospheres.

The tests were made using the following weights of material:

Garbage	0.80 to 1.00 g.
Coal and cinders	0.50 " 0.75 "
Rubbish	0.35 " 0.50 "
Fine ash	0.50 " 0.75 "
Clinker	0.50 " 0.75 "

These values, as obtained by the actual tests, were reduced to values per pound of dry sample, original sample, and combustible, in that order, using the corrected proximate analyses as a basis. These are the figures that appear in Tables 6 and 7.

There was no difficulty in securing satisfactory combustion, except in the tests of "fine ash" and "clinker," in which the percentage of inert matter was so high that it prevented ignition of the combustible portion of the sample by the ordinary means. In these cases, therefore, a small amount of naphthaline was introduced with the sample to start the combustion, and a deduction, representing the calorific value of the naphthaline used, was made subsequently.

The residue from the combustion of the garbage was hard, vitreous, and invariably in the form of small globules of a brownish black color. That of the coal and cinders was naturally about the same in appearance as the ash of anthracite coal, while the rubbish left little more that could be seen with the eye than a stain on the combustion tray.

At the beginning of the experiments, tests were made in duplicate

Mr. Welton. on all samples until it became evident that the differences in results, as shown by the duplications, were well within the variation that might easily occur in the primary selection of representative samples. In amount, these differences were generally less than 1% of the calorific value of the dry material. By this time, also, the uniformity in the character of each class of material, as shown by the calorific value per pound of combustible, began to be noticeable, and it was observed that this value would serve to detect errors in manipulation and computation as well as to indicate the occasions when duplication was required. As a consequence, tests on the same sample were rarely repeated thereafter, unless the value per pound of combustible was at some variance with the average of the other tests already made.

To those who are not familiar with the calorific values of the staple fuels, such as anthracite and bituminous coals, it may appear that no great confidence should be placed in the results of these tests on material which would naturally be expected to vary widely in character. As a matter of fact, the experiments have shown a uniformity of character in the material which is all the more remarkable in that it was not anticipated. Indeed, now, when all the data are at hand, the conclusion might easily be drawn that in the instances where the largest variations in calorific values per pound of combustible occur, this variation is more likely to be due to the difficulty of obtaining representative samples from the collections than from actual differences in character.

Moreover, few who have had no occasion to study the matter of analyses and calorific tests of coal are aware of the variation in fuel value of its combustible portion or what is known as "pure coal."

In this respect the figures in Table 12 are of interest. These are deduced from:

First.—The report of the coal-testing plant of the United States Geological Survey at St. Louis, in 1904;

Second.—From records of the Department of Water Supply, Gas and Electricity, at Mount Prospect Laboratory, New York City.

The chemical analyses made by Professor Peckham consisted of organic analyses of composite samples representing the collections of the entire period, and inorganic analyses of the residue from burning the same over a Bunsen flame. They will not be described in detail here, but the results of both series of analyses have been combined in Table 13.

These results would have been included in Mr. Fetherston's paper, but for the fact that their completion was delayed by pressure of more important matters in the laboratory, and they have only very recently become available.

TABLE 12.—Range of Calorific Values, Shown by Tests, and the Proportion Such Range Bears to the Average Values.

Basis.	Anthracite Coal.		Bituminous Coal.				Municipal Refuse (1906).					
	Department of Water Supply, Gas and Electricity, New York City.		U. S. Geological Survey.				Laboratory, Commissioners of Accounts.					
	From Pennsylvania, (56 samples).		From West Virginia only, (12 samples).		From West Virginia, Pennsylvania, Illinois, and Iowa, (23 samples).		Coal and cinders, from Borough of Richmond, (24 samples).		Garbage, from Borough of Richmond. (23 samples).		Refuse, from Borough of Richmond.	
	Range, in British thermal units.	Percentage of average value.	Range, in British thermal units.	Percentage of average value.	Range, in British thermal units.	Percentage of average value.	Range, in British thermal units.	Percentage of average value.	Range, in British thermal units.	Percentage of average value.	Range, in British thermal units.	Percentage of average value.
On the basis of the calorific value per pound of combustible	1 080	7.5%	879	5.7%	3 458	23.8%	1 960	13.7%	2 220	21.5%	1 640	19.0%
On the basis of the calorific value per pound of dry sample	6 379	50.0%	1 464	10.0%	5 342	42.0%	5 155	60%	4 265	51%	3 053	42%

Mr. Welton.

The same reason also accounts for the relatively large percentage of undetermined constituents in the garbage sample.

Table 13 also shows a comparison between the calorific value of the samples, as calculated from the chemical analyses, and as determined by the calorimeter. The correspondence is believed to be sufficiently close to serve as a general verification of the entire work. The percentages in Table 13 are all computed on the weight of dry samples as a basis. The presence, in considerable amounts, of volatile hydro-carbons in the garbage and rubbish samples may be noted if the carbon, as shown by the chemical analyses, be compared with the fixed carbon of the proximate analyses reduced to a dry-sample basis.

In the determination of the moisture in the garbage samples for the proximate analyses, it is extremely probable that some of the lighter and more easily volatile hydro-carbons were driven off and computed as moisture. This is undoubtedly the reason why there is not a closer agreement between the calorific values of dry samples of garbage, as calculated from the analyses, and as determined in the calorimeter. The oxygen is determined by difference, and the ash is the average of the proximate analyses reduced to the dry-sample basis.

TABLE 13.—Chemical Analyses of Dry Composite Samples of Coal and Cinders, Garbage, and Rubbish, Representing Collections for the Year 1905-06.

Constituents.	Coal and cinders.	Garbage.	Rubbish.
Percentage by weight of:			
Carbon	55.77	43.10	42.39
Hydrogen	0.75	6.24	5.96
Nitrogen	0.64	3.70	3.41
Oxygen	2.37	27.74	33.52
Silica	30.01	7.56	6.49
Iron oxide and alumina	8.98	0.41	2.03
Lime	1.21	4.26	2.26
Magnesia	Trace.	0.28	0.57
Phosphoric acid	None.	1.47	0.10
Carbonic acid	None.	0.59	1.49
Lead	Trace.	Sulphides, 0.20	0.52
Tin	Trace.		Trace.
Alkalies and undetermined	0.27	4.45	1.21
	100.00	100.00	100.00

Calorific Values, in British Thermal Units.

Calculated from above analyses	8 382	7 970	7 250
Average of calorimeter determinations	8 510	8 351	7 251

The calculations from the chemical analyses are made as follows:

$$62\,100\left(\mathrm{H}-\frac{\mathrm{O}}{8}\right)+14\,500\ \mathrm{C.}=\text{British thermal units.}$$

C. Herschel Koyl, Esq. (by letter).—This paper presents in admirable form the results of a very careful, systematic, and thorough study of the possibility of destroying by fire the mixed wastes of the Borough of Richmond in a manner innocuous, inoffensive, and not too costly. Mr. Koyl.

The need of such an investigation was pressing, and its value not merely local, because the number of small communities in America, in which this problem is of first importance, is considerable and growing.

The technical question is whether the mixed waste contains enough combustible to be self-burning, at a temperature sufficiently high to destroy and not merely distil the volatile organic matter. Records show that, in England and on the Continent, a satisfactory disposal of mixed municipal refuse is made in this way, but it is also known that abroad there is less waste of edible matter than in the United States; and, therefore, before risking $60 000 of municipal money, it was the part of wisdom to determine the theoretical fuel efficiency of the waste of Staten Island. From the character of the examination and its completeness, Mr. Fetherston's results may be accepted with confidence, and also his conclusion that a destructor of the English type will burn the mixed waste of the Borough of Richmond effectively. True, the expense will not be small; but if the destruction of organic matter is complete and inoffensive to the neighborhood, a cost of from $1.00 to $1.50 per ton should not be prohibitive, in view of the fact that any other method of disposition would be extremely difficult in Staten Island.

Regarding the limits of usefulness of these waste destructors: It has been proven by eight months' operation in Westmount, Montreal, that an average of 20 tons of mixed waste per day can be destroyed at a working cost of 31 cents per ton, and a total cost of 80 cents per ton; therefore, a population of 13 000 people is not too small to have the economical service of a destructor.

An upper limit, however, is reached when considering a city from which there is enough garbage to make profitable a modern reduction plant to separate the organic matter into grease and fertilizer. For instance, the City of New York makes satisfactory disposal of approximately 3 000 000 tons of mixed waste per year (70% ashes, 12% street sweepings, 12% garbage and 6% light refuse, by weight) at an average cost of 40 cents per ton. It would be folly to talk of putting all this material through destructors at a cost of 75 cents per ton.

Note should be made of a fact not mentioned by Mr. Fetherston, that in the "coal and cinders" which makes 27% of his total collection, or about 35% of his ash collection, more than half is not only burnable coal, but salable coal. This arises from the fact that most of the coal which gets into the ash-pit is undersized for the grate and

Mr. Koyl. falls through unburned and indeed unmarked by the fire. The writer has taken from many sample tons of Manhattan ashes an average of 20% of salable coal, from furnace size down, of which about half, after being washed, was indistinguishable from coal fresh from the mine. This means nothing in the Borough of Richmond, but it will be the determining factor in settling the method of final disposal where anthracite is used in cities which, at the same time, are large enough to make profitable the mechanical separation of the coal from the clinker and ashes. The process is not more difficult than the concentration of ore, and there is an average profit of about $2 per ton of recovered coal.

In the United States the advocates of reduction "utilize" garbage by separating it into water, grease, and fertilizer. The advocates of incineration "utilize" dry refuse by picking out its 30% of salable paper, rubber, etc., before they burn the remaining 70% of rubbish. And "utilization" is the keynote to successful policy in any large city.

It now costs New York $1 250 000 for the final disposition of its municipal wastes. It would cost $2 250 000 to put all the waste through destructors. It would cost about $200 000 to do it scientifically and save what ought not to be burned or buried, as follows:

			Cost.	Profit from utilization of rubbish and coal.
Garbage	360 000	tons (contract)	$200 000	
Street sweepings	360 000	" at 40 cents	144 000	
Ash and clinker	1 680 000	" at 40 cents	672 000	
Rubbish	180 000	" burned at profit	.....	$40 000
Coal	420 000	" recovered at profit	.....	840 000
			$1 016 000	$880 000
Less			880 000	
			$136 000	

or, the Department of Final Disposition would be almost self-supporting.

Mr. Tribus. Louis L. Tribus, M. Am. Soc. C. E.—This paper gives evidence of a great deal of work, and the speaker can say from personal knowledge that, in the Borough of Richmond, and along the lines described, there has been a vast amount of work which does not appear in the paper, yet its results will certainly secure great advancement in the art of refuse disposal.

Prior to the inauguration of the Greater City of New York, Staten Island (then becoming the Borough of Richmond) was occupied by a

number of corporate villages and a great many small hamlets, the latter controlled by the usual "township" and "county" system of government, the incorporated portions by "village" form, with more or less intelligent management, as politics determined. Mr. Tribus.

During the first four years following consolidation, little was done, other than to get accustomed to being a part of the great city.

On January 1st, 1902, under the revised charter, considerable home rule, and a borough president of character and ability, the first advance toward real progress was made.

Street cleaning and refuse disposal had been cared for, to a very limited extent, during the preceding four years, with a small force of men, but supervised by a man trained under Colonel Waring, who had the welfare of his subject at heart. The speaker was called upon by the borough president early in 1902 to act in both professional and executive capacities, and take charge of the public works and maintenance bureaus of the borough. He was given very free rein in securing betterments in plan and operation, but, on taking charge, insisted that he was to be free from politics in the work itself. Richard T. Fox, formerly in charge of the work in Richmond, as noted before, for the Department of Street Cleaning for the whole city, was placed in charge of the newly created "Bureau of Street Cleaning," which at this time came under the President of the Borough through the Commissioner of Public Works. Fairly liberal appropriations were made, so that, after careful plans had been laid, improvement became the order of the day. The first step noted was the banishment of garbage cans and refuse receptacles from the sidewalks and streets as far as possible, all such being removed by collectors from behind the buildings, the empty cans being then returned to their places. The next step toward efficiency and the establishment of *esprit de corps* among the men, was made by placing them all in uniform; the third step was to employ the men continuously throughout the year, so as to render service daily instead of spasmodically. This, of course, applied more to street cleaning, pure and simple, than to refuse collection which, formerly also, had to be more or less regular throughout the year.

After some two years' service, Mr. Fox accepted a call to the City of Chicago, to show there what scientific and business methods could accomplish in the way of street cleaning and refuse disposal.

Mr. Fetherston, as a member of the borough engineer corps, had been assigned to specific work in connection with local scientific tests in refuse disposal, in which work he gave so good an account of himself that when Mr. Fox resigned he was selected to take the place of "Superintendent."

The paper describes very clearly the course taken, which has led to the recommendation and the actual construction of the first refuse

Mr. Tribus. destructor of this type in the United States. It is confidently hoped that in a few years this paper will be supplemented by one describing the destructor and telling of its efficient operation. That, however, will depend largely on the intelligence exhibited in its management, for the best piece of machinery may give poor results unless well handled.

In studying the refuse disposal question in the Borough of Richmond, it has been necessary to estimate very carefully the probable development of this specific locality, as its conditions are changing very rapidly year by year. It is not improbable that, within the life of the present generation, the whole island will be practically built up with residences, factories, stores, and valuable water-front improvements. This would mean that it would be impossible to find places for the burial of garbage, for maintaining ash dumps, and for any of the nuisance-producing plants for the destruction of garbage by low-temperature cooking. All experiments, therefore, in the past six years have been directed toward finding a process that would convert refuse, without nuisance, into some useful or innocuous material. The investigations which have been made so carefully by Mr. Fetherston and others assigned to the work from time to time, therefore, have been directed specially to this system; as, by process of elimination, all other systems were dismissed as not suitable for the probable local conditions of development, hence the conclusion that mixed refuse destruction promised more to the Borough of Richmond than did any other process; though it should be clearly understood that other systems might be more advantageous in other communities under different conditions, and time may prove that even in Richmond some different method may be evolved. In view of these explanations, the special studies, almost exclusively, have been directed toward acquiring information about and perfecting plans for mixed-refuse destructors, with the primary object of collecting materials at the lowest expense and converting them into an innocuous product without causing nuisance in the process, and it seems probable that the high-temperature system planned will accomplish the object desired. Up to the present time, theory and experiment indicate that, not only will the material collected have sufficient fuel value in itself to convert it into inoffensive slag, but that, in addition, there will be developed a liberal amount of heat units.

In the installation under construction, there is provided a boiler which, it is expected, can be operated by the otherwise wasted heat units, so as to furnish ultimately all the power and light needed at the plant. If the results justify the expectations, it is probable, also, that the slag from the destructor will eventually be ground up, and, with an admixture of cement, be converted into paving blocks, which would have value for gutters and pavement in places where traffic is

not specially heavy. That feature, however, is only being considered for the future, the present epoch being confined to what might be called the self-destruction of the refuse collected. Mr. Tribus.

If success attends the operation of the new plant, it is expected that, ultimately, six or seven similar plants will be installed in other portions of the borough, as near as possible to the centers of collection; with one concession, however, to public sentiment, placing the destructors in manufacturing districts, near a railroad, or at the water front, rather than in residence localities.

Mr. Fetherston's paper covers the general phase of the refuse destructor, from the standpoint of economy in gathering garbage and other refuse and disposing of it finally. He has not gone particularly into the reasons why prompt final disposal of refuse is desirable. There are, perhaps, three reasons why every community should take care of this feature of urban life: First, that which appeals most popularly to citizens, the removal of refuse materials because they are obnoxious to the senses of smell and sight; second, because the keeping of decaying organic matters near habitations is generally supposed to breed disease; and, third, a reason which should have more consideration, though it has not been taken up extensively, that, during the heat of the summer, when the house fly develops and feeds and thrives on refuse, it is a very prolific distributor of disease. It is, to begin with, not a cleanly insect, and feeds on decaying matter; then, as likely as not, it proceeds to the nearest receptacle containing milk, for a drink, and not infrequently a bath; the combination is often too much for the fly, and it remains for a few moments or several hours floating around in the milk, leaving in it very often the germs of disease, which in turn thrive very readily in the milk and are taken into the human system. During the summer, the human system, particularly in infancy and childhood, is in excellent condition for the growth of disease germs in the intestines, and the various so-called summer complaints ensue. While probably no one as yet will claim that all intestinal diseases are caused by flies; by the process of elimination, in records that have been kept in certain places by intelligent observers, the fly can very fairly be charged with a great deal of the trouble. The mosquito has borne its share of public contumely as a dispenser of yellow fever; why should not the ordinary house fly be given credit for the work which undoubtedly it can do, and which many are beginning to believe it does do? If this is the case, the community that promptly removes and disposes of its decaying organic matters should, first, enjoy the presence of a lessened number of flies, and, second, a lessened number of cases of intestinal disease. This subject is only mentioned here as one worthy of fuller investigation, rather than as a conclusion based upon observation.

This whole topic of refuse collection and disposal is one of very

Mr. Tribus. great interest, and is a field as yet almost untouched in the United States, and, prior to the publication of this paper, very little, of much real value and based on facts, has been printed. The speaker hopes that additional information will be gathered, not only in the Borough of Richmond, but in other places, and be put at the disposal of this Society, to aid in this most important work.

Mr. Leask. H. Norman Leask, Esq.—The speaker, being conversant with the literature on this subject, and having had long experience in designing and operating destructor plants, ventures the opinion that, for engineers, this is one of the most valuable papers which has ever been presented. It is the more valuable as it enters a new field and presents data from which a contracting engineer can design plants and guarantee results without risk of failure to either of the contracting parties.

It is pleasing to note that the author has commenced with first principles, and not at some place in the middle of the subject, which, unfortunately, is often done. The exact figures in the paper, however, do not apply generally, and must be used with the discrimination born of experience, due allowance being made for losses, after the manner set forth in the problematic balance sheet, Table 9.

The information in the paper has not been available heretofore in such extended form; and, as far as the speaker is aware, it has not covered such a long period, or such a great weight of refuse. At the same time, it should be remembered that an inspection of material is desirable, in order to note any peculiarities in its character, without which immediate success is not likely to result.

When the author took up the question, the conditions existing in the Borough of Richmond were practically the same as those which have induced most cities to resort to destruction by fire. In many cities abroad other methods have been tried, such as reduction, making fertilizer, and gasification for lighting and power purposes, all of which have failed signally to deal completely and in a satisfactory manner with the final disposition of refuse of all classes, which is the chief desideratum. The only system of final disposal which is growing in use, and is now quite general, is the destruction or cleansing of refuse by fire, thus rendering it innocuous.

That an examination of existing garbage crematories in the United States should offer no hope of meeting the requirements satisfactorily, is not surprising, for such crematories can hardly be termed engineering propositions, and one doubts very much whether American engineers have had anything to do with their design. The principal faults which one recognizes in garbage crematories of the present type are:

1.—That the process of destructive distillation, rather than oxidization, has been resorted to.

2.—That apparently no lower-limit temperature has been regarded as a standard by the builders of such apparatus,

Mr. Leask.

although it is absolutely necessary to maintain a temperature of more than 1 250° fahr., in order to insure the combustion of the hydro-carbons and the dissociation and oxidization of objectionable chemical compounds.

3.—That the usual method of feeding and stoking precludes the possibility of obtaining anything like a regular temperature in the furnace, the temperature rising and falling with an amplitude of probably 800°, and sometimes falling as low as atmospheric temperature.

4.—That the high temperature is usually at the wrong end of the furnace, namely, that farthest from the outlet, and as long as this remains there can be no hope of dealing successfully with the material in a sanitary manner. Attempts to overcome this difficulty have been made by following the ideas of Mr. Charles Jones, of London, who, in 1885, introduced a fume cremator between the furnace and the chimney. This was a palliative rather than a cure, and, while it succeeded in reducing the nuisance to some extent, it only went half way.

5.—Another error, in certain types of garbage cremators, is made in the environment of the burning mass. Water-jacketed furnaces are absolutely unsuitable for burning garbage or other material high in hydro-carbons. In such a furnace, flame is no sooner generated than it is extinguished by absorption, due to contact with cold surfaces, or by radiation. No one would think of hatching eggs out in an ice box.

6.—Finally, restrictions as to the amount of organic matter remaining in the ash after cremation do not seem to be imposed upon the builders of such apparatus, nor have such apparatus succeeded satisfactorily in eliminating the organic matter from the ash.

Table 1 is likely to give a very erroneous impression as to the quality of the refuse collected in Great Britain. That the author does not rely on the figures given in this table, is quite apparent. His estimates of the character of the refuse in various cities, as given in Table 10, prove that the conditions at the plants he visited did not correspond with the figures in this table. Hutton's figures as to the percentage of coal, coke, breeze, and cinder are much too high, even for mid-winter. Mr. Codrington's figures appear to be much nearer to the actual conditions, while those of Mr. Russell, giving 64.53% for coal, coke, breeze and cinder, are not justified by the results which he has obtained at the Shoreditch plant, the operation of which he directs. The figures for Torquay appear to be inverted, and,

Mr. Leask. if inverted, would more fairly represent the conditions existing in that town and in similar towns along the south coast of England and in suburban districts.

That there is a marked similarity between the refuse collected in Great Britain and that collected in New York is undoubted, and the difference relates more to character than to calorific value. The speaker agrees with the author that there is probably more moisture in refuse, as collected in Great Britain, than in refuse collected in the Borough of Richmond, but it is in a different form. In the Borough of Richmond the moisture is principally contained in the garbage, while in Great Britain the ash, as a rule, contains quite a large percentage of water, and it must be remembered that, in this form, it is more easily attacked than when carried in the structure of material such as garbage.

There is a similarity, also, between the refuse collected in many cities on the Continent of Europe and that collected in the United States. It has been stated that the refuse collected in Berlin has a calorific value of about 2 000 B. t. u., while at Frankfort it has a calorific value of 4 350 B. t. u.; the refuse collected in Vienna is stated to contain about 3 000 B. t. u., and that at Kiel somewhat less. The refuse collected in Paris has been analyzed frequently, and has been variously stated to contain from 3 600 to 5 400 calories.

It should be noted that the chemical analysis of the refuse at Kings Norton was made of refuse collected in winter. In the spring, three years later, another analysis was made, and the refuse was found to contain 4 300 B. t. u. In summer, however, the calorific value could not be much more than 3 000 B. t. u.

It might be interesting to give the calorific value applied to various classes of refuse by German scientists, in order that a comparison may be made between that part of Table 1 devoted to that subject and the values ascertained by Mr. Welton:

Vegetable matter ...	2 165	B. t. u.
Bones	540	"
Paper	3 950	"
Sawdust	5 750	"
Wood	6 280	"
Straw	5 400	"
Coal, coke	9 380	"
Hair	1 620	"
Rags	3 600	"

It will be seen that the calorific value of vegetable matter corresponds very closely with Mr. Welton's figures, while that for coal, coke, etc., is somewhat higher, and that for rubbish (composed of paper, wood, rags, etc.) is appreciably lower.

Mr. Leask.

In order to make a comparison of the refuse collected in the Borough of Richmond and that collected in the London residential district, containing some stores, and a suburb of one of the large provincial towns, the speaker's firm, by the courtesy of the city engineers, made a number of analyses of the refuse as collected in the Metropolitan Borough of Stoke Newington and Kings Norton, near Birmingham. This refuse was sorted by hand into four classes:

1.—Garbage;
2.—Coal, coke, cinders and fine dust, including fine inseparable vegetable matter;
3.—Rubbish;
4.—Large incombustible matter, such as tin, bottles, etc.

These analyses were made at a period corresponding to the critical month in Richmond, that is, September and early October. The volume of the refuse at Stoke Newington worked out to about 4 cu. yd. per ton of 2 240 lb., and it contained on an average: 34.43% of garbage, 42.92% of coal, fine dust, etc., 15.4% of rubbish, and 7.35% of glass, metals, etc. The refuse had averaged 4 cu. yd. to the ton for about five months, and presented somewhat similar characteristics during this period. At Kings Norton the refuse had a volume of about 3.75 cu. yd. per ton, and carried 39.5% garbage, 45.4% coal, fine dust, etc., 9.3% rubbish, and 5.2% glass, metals, etc. Taking September alone, there was 49.37% garbage, 38.80% coal, coke, etc., 7.73% rubbish, 4.28% glass, etc., from which it can be seen that the percentage of garbage is even higher than that collected in the Borough of Richmond for that month. It is not suggested that the foregoing figures are absolutely accurate, but merely the result of an honest endeavor to ascertain the make-up of the refuse. The results obtained with the refuse at Kings Norton agree very well with the balance sheet, as shown in Table 9, as to evaporation and combustion-chamber temperature. In September the evaporation was somewhat higher than that calculated in the balance sheet, that is, more than 1.25, actual, in ordinary work, and the average temperature in the combustion chamber about 150° higher. The speaker's make-up of a balance sheet would differ slightly in that the radiation loss would not be as high, while there would be a somewhat lower percentage of unburned carbon in the clinker and ashes, but probably a greater loss in moisture in chimney gases. As the average combustion-chamber temperature and evaporation in winter and spring are considerably higher than the foregoing, it can be seen that, in British refuse, as well as that under discussion, there is a considerable seasonable variation.

The practical tests, as given in Table 8, demonstrate clearly that the material is burnable, and the results obtained are such as might be expected when burning material in such a crude furnace, and pre-

Mr. Leask. clude all doubt of obtaining satisfactory temperatures with a properly regulated and heated air supply.

Based on the results obtained with summer refuse in England, Messrs. Heenan and Froude, Ltd., of Manchester, who have been entrusted with the plant for New York City, specially designed and erected a plant in Vancouver, B. C. This plant has certain departures from their standard type. It is now in successful operation, and, of the refuse collected in that city, it is destroying more than 50 tons per 24 hours, at suitable temperatures, without the aid of supplementary fuel, and with an excellent residual. The refuse burned is very poor in quality, due to the presence of considerable quantities of wood-ash and moisture. The difficulties of obtaining high temperatures with this material have been overcome by checking the quantity and increasing the temperature of the air supply. Figures in detail as to the results are not yet at hand, therefore they cannot be given here. The cost of destruction is 36.1 cents per ton, including the salaries of two engineers who look after the pump, boiler, fan, etc. During the first week the plant was put in operation, it surpassed the guaranty, which is unusual—the men being untrained—and better results may be expected in the course of time.

The utilization of the steam and clinker resulting from the destruction of refuse is by no means the only offset to the cost of burning. The most important offset is the reduction of the cost of collection, for a modern plant may be placed in the center of a city or residential quarter without fear of nuisance to the neighborhood. This means a great reduction in the cost of collection and transport. Numerous instances can be cited supporting this: in the Metropolitan Borough of Stoke Newington, previously mentioned, two 45-ton units, each with 200-h.p. boiler and the appurtenances thereto, a clinker-crushing and screening plant, has been erected in the middle of the Borough, at the rear of the Town Hall, and surrounded on all sides by three and four-story dwellings of a good class. Notwithstanding the fact that the interest and sinking fund on the capital outlay, the repairs, maintenance, and labor charges have to be added to the cost of disposal, the cost of collection, transport, and final disposal is now lower than it was prior to the erection of the plant, and this in spite of the fact that, as yet, no use has been made of the steam generated, which is equivalent to about 175 k.w. per hour from one unit; also exclusive of the sale of clinker, which has been contracted for on a profitable basis. The same conditions apply to Woodgreen, London, where the plant is also critically located, and at Rathmines, Dublin, where a saving in coal of $2 000 per year (in addition to the saving in cost of collection and transport) has been made, and where, in the past, the power has been utilized without relying on storage batteries.

Mr. Leask.

Now that storage batteries have been installed, the saving in coal, as shown by the working during the past few months, will be more than $5 000 per anum. This is the more remarkable as the quantity of refuse to be handled is less than 35 tons per day.

Table 9 gives the average temperature which may be expected in the combustion chamber at various seasons; it gives no idea, however, of the lowest temperature which may occur. When burning September refuse on the standard British furnace with air heated to, say, 250° fahr., the lowest temperature would be near, if not actually below, the limit of 1 250°, momentarily. To insure the temperature always being above this lower figure, it is absolutely necessary to increase the temperature of the preheated air, to control the air supply very carefully, and, further, to increase temporarily the temperature of the air entering the furnace immediately after a fresh charge. Fortunately, this can be effected by taking the heat out of the clinker, just withdrawn from the grate, prior to charging. All these points have been given special attention in the case of the plant for New York City.

British destructors have been designed in accordance with the principles mentioned by the author, but it has been by progressive steps, and after many failures. The various steps in the conception and improvement of refuse furnaces, as made in England, may be traced as follows:

1.—The attempt to burn refuse in or under shell boilers;
2.—The building of a fire-brick lined furnace, or Dutch oven, operated by natural draft;
3.—The introduction of the fume cremator;
4.—The abandonment of the fume cremator and the introduction of forced draft;
5.—The preheating of the air supply;
6.—The use of a continuous furnace chamber, containing a number of grates with divided ash-pits;
7.—Suitable ventilation of the building;
8.—Methods of handling the clinker and recovering the heat contained in it.

With regard to the cost of operation, it is possible, with a large plant, to reduce the labor charge, part of the work being effected by mechanical means. It must be remembered, however, that the feeding of the furnace is only one of three operations necessary in the working of the plant: First, there is the introduction of material into the furnace, which may be done mechanically; the other two, which, however, do not appear to be susceptible to mechanical operation, are the stoking and spreading and the final cleaning out of the mineral residual from the grate.

Mr. Leask. The speaker's firm has attempted to solve this problem, and after many failures has at last succeeded in devising a machine which will handle all classes of refuse and will feed the refuse in any desired quantity. Extended trials of such an apparatus have been made at one plant, and it will soon be installed in some city. The system to be adopted for charging depends on the specific gravity of the material to be dealt with and the size of the plant.

The speaker has had the opportunity of examining the refuse in a number of large cities in the United States, and is strongly of the opinion that the combined refuse of most cities can be destroyed by fire at suitable temperatures, without the aid of supplementary fuel.

This brings up another phase of the question which has been mentioned by Mr. Fetherston, namely, the collection of the refuse. The adoption of a system of a single collection of refuse, combining the ashes, rubbish, and garbage, cannot be urged too strongly. It is impossible to get complete or satisfactory separation. It is a question of public health, rather than profit. The single collection costs less to make. The mixing of the refuse retards decomposition, the ashes acting as a deodorant by absorption, and it provides a wherewithal in calories to cleanse the mass of its impurities, and, when burned, leaves a marketable residual in the form of clinker. If advantage be taken of the heat generated by combustion (and here it should be noted that, whatever the material may be, when it is burned at suitable temperatures there is always utilizable heat), there is placed at the disposal of the authorities another valuable residual in the form of steam, the best uses for which are those giving a large load factor, such as pumping—sewage or water—or electric traction. Lighting alone is not a satisfactory outlet. The question of collection and disposal of refuse in the United States to-day appears to be in the state that it was in older countries some years ago, that is, in the hands of contractors. It has been abundantly demonstrated that the only satisfactory method is for the municipalities themselves to undertake it. Where this has been done, and where refuse destructors form a part of the scheme, it has been followed by a noticeable decrease in the death rate. Why should not one of the richest countries in the world forsake the problematical gain arising from reduction, and regard the question from a purely public health standpoint, as has been done in older countries, even in backward Russia?

The whole question is purely one of combustion, and, generally speaking—provided the moisture contained in the material is not excessive—refuse containing 2 000 B. t. u. per lb. will cleanse itself.

The great difference between the combustion problem as applied to coal and refuse is this: When dealing with coal one has a material which is comparatively low in ash and requires about 20 to 24 lb. of air per lb. to burn it in a practical and satisfactory manner, whereas

refuse is high in ash, and the air required is only from 4 to 5 lb. per lb. The difficulty, therefore, is to find a small quantity of carbon in so large a bulk, with the minimum quantity of air. To effect this one must look to the distribution and the temperature of the air supplied, the intensity of the draft, and the environment of the burning mass. It is, therefore, wholly an engineering proposition. Mr. Leask.

The speaker must again congratulate the author on the service he has rendered to engineering science in general and contracting engineers in particular.

RUTGER B. GREEN, M. AM. SOC. C. E. (by letter).—A possibly interesting addition might be made in the future to Mr. Fetherston's complete statement of conditions to date, by experimenting with the drying of garbage in kitchens before it is turned out for collection by the city wagons. There is usually a great deal of heat wasted in a kitchen stove that might be used in this way, particularly if some simple hand-press was first used to squeeze out the free water. The effect should be to retard decomposition, making less frequent collections practicable. It would also get rid of the flies and the unsightly offensiveness of the drippings from garbage cans and collection wagons. It seems possible that a little competent supervision, backed by an ordinance, might easily educate householders into turning out their garbage dry enough to be wrapped in coarse brown paper. A swill wagon would then be no more offensive than a grocer's wagon, and a refuse disposal plant in a neighborhood no more objectionable than a butcher's shop. Mr. Green.

This change of sentiment could be encouraged by the utilization of the waste power from the refuse furnaces in the commercial heating of stores and houses in the neighborhood. The writer has for three years enjoyed the service of a commercial heating company in his home. Merely turning steam on and off, like gas, water, and electric light, seems to be so great an improvement over the dust and bother of buying coal, keeping the furnace going, and getting rid of the ashes, that it is thought the day will soon come when individual furnaces will be as rare as tallow candles, rain-water cisterns, or cesspools. Houses able to command this convenience should be at a premium over those too far from a disposal plant to be heated from it. Further, there would be no ashes to be collected from a district commercially heated. Mr. Fetherston states that electric lighting is objectionable as a means of utilizing the heat from destructors, because light is needed only a few hours a day. Commercial heating should overcome this difficulty by furnishing an all-day demand.

GEORGE N. COLE, ASSOC. AM. SOC. C. E. (by letter).—In Chicago there is a refuse disposal plant which has been conducted successfully as a private enterprise for many years, and the writer thinks it should, at least, be mentioned, because the existence of such plants, if at all Mr. Cole.

Mr. Cole. extensive, changes the whole problem of municipal refuse disposal. Private plants of this kind take their supplies from practically only one of the classes into which Mr. Fetherston has divided all refuse.

It is regretted that only a superficial description of this plant can be given, and it is hoped that one of the Chicago members of the Society will gather the details for what would prove to be quite an interesting paper.

The writer's interest in the plant was aroused because it was such a marked example of reclamation. The proprietor took what other people were glad to get rid of, and coined it into a mintage of six- and eight-story buildings, which only increased further his coining ability.

The plant is in the heart of Chicago's factory and loft district. It is near the South Branch of the Chicago River and west of where the Metropolitan Elevated Railroad crosses that river, being just under the north side of the elevated structure.

It is operated as a part of the loft and power buildings, and consists of a boiler-room of good size, with furnaces of the kind used for burning shavings, a storage room holding about a couple of carloads of refuse of various sorts, a small coal-bin, and a large engine-room. At the time the writer saw this plant it was furnishing sufficient power to operate machinery in the power lofts of six- and eight-story buildings, covering about a full city block, to which more have since been added.

The material was, in a way, of a selected nature. No wet stuff or garbage was received, but considerable quantities of stable refuse were taken. For the most part, the material consisted of shavings, paper, leather, and the burnable refuse that is cast out of small shops of all varieties.

The writer was informed that the supply of this refuse was very uniform and regular. In case the supply gave out, coal was burned, and, if there was no room for loads, the drivers were paid 50 cents to carry them to the filling dump on the Lake Shore. The writer was also informed that the quantity of coal used would seldom amount to 12 tons per month, being generally much less, and that very little money was paid for hauling the over-supply. The writer knows nothing as to the ashes and their disposal. The power was transmitted at that time by hemp rope. The smoke from this plant is an improvement on the usual output of that locality.

The study of a plant of this kind may lead to valuable information. There are places for such installations in all cities, unless, of course, the city pre-empts all the refuse in order that the dry may aid in the disposal of the wet. At any rate, the increase in the plant referred to shows them to be commercially feasible, and yet their existence will disturb municipal plants very seriously and decrease their ability to burn garbage.

Mr. Fisher.

EDWIN A. FISHER, M. AM. SOC. C. E. (by letter).—For more than three years prior to September 1st, 1906, the writer consulted every available published work or report in the English language dealing with the disposal of municipal wastes, and he takes pleasure in saying that the work of Mr. Fetherston is the most thorough and painstaking of any he has found. There is no branch of municipal work to which the saying of Artemus Ward, "It is better not to know so much, than to know so many things that ain't so," more aptly applies. A large portion of the investigations of this subject heretofore made have been conducted by council committees without expert advice, therefore they have dealt in generalities rather than in detailed information.

Early in 1904 Mayor Cutler requested the writer to make an investigation of the subject of the collection and disposal of garbage and refuse for the City of Rochester, N. Y., together with recommendations for a better method of collection and disposal than was then in use. At that time the garbage was being collected under a yearly contract, which included both collection and disposal. The contractor disposed of the garbage to the farmers of the outlying towns, or made arrangements with them to dispose of it on land outside the city. The health authorities of the adjoining towns objected to this method of disposal, and it became evident that some other and more sanitary method must be adopted.

The writer visited a number of plants in the United States and made a careful study of the recent literature on the subject of garbage and refuse disposal in other cities, both at home and abroad. Near the close of 1904 it became evident that more time would be required in order to make an intelligent report, and the writer recommended that the same method of disposal as provided in 1904 be continued for another year.

Near the close of 1905 a similar condition existed, and another yearly contract was entered into.

The estimated population for 1904 was 175 000. The total quantity of garbage collected for the year was 22 964 tons, or an average of 0.72 lb. per capita per day. For 1905 the population was, by the State enumeration, 181 670; the total quantity of garbage collected was 21 800 tons, and the average per capita, 0.66 lb. per day.

In September, 1906, the writer completed his report, and made the following recommendations:

Eventually the city will dispose of the garbage and refuse by burning, and the collection and disposal will be done directly by the municipality. In view, however, of the present unsatisfactory conditions of this method of disposal in the United States, it was suggested that its operation be further postponed, and a contract let for the collection and disposal of garbage, pursuant to the amendment of the charter,

Mr. Fisher. for a period of 5 years from January 1st, 1907. It was recommended that the collection for the central portion of the city should be made daily and for the remainder of the city, from May 1st to October 1st, three times a week; and, for the remainder of the year, weekly. It was also recommended that, as all of the reduction methods of disposal are subject to interruption from the failure of some portion of the plant, the contractor should procure, either by lease or purchase, a sufficient area of land to bury the garbage without offense to the surrounding property, in case of a breakdown in the disposal plant. It was also recommended that the city have an option to take over the plant at a valuation to be fixed by arbitration at the end of the term.

It was further recommended that the contractor should satisfy the Board of Contract and Supply that the process he intends to adopt has been proved to be successful by actual operation under contract in some city of considerable size. He should furnish sufficient general plans and descriptions of the plant so that the Board may satisfy itself that his equipment is sufficient to take care of the maximum quantity of garbage and dead animals that may be required to be disposed of at any time during the term of the contract.

The writer also further recommended a separation of the ashes and rubbish, and the construction of a suitable plant within the city for the disposal of the entire rubbish by burning; the plant preferably to be constructed and operated directly by the city.

The recommendations of the writer with reference to the garbage collection and disposal were adopted, and a contract was entered into with the Genesee Reduction Company for the collection and sanitary disposal of the garbage and dead animals in the City of Rochester for a period of 5 years from January 1st, 1907, at a yearly compensation of $59 770. The estimated quantity of garbage for the 5-year period from January 1st, 1907, to January 1st, 1912, was an average of 83 tons per day. The contractors bidding for this work, including the successful bidder, insisted that this was very much in excess of what they believed would be the actual quantity. It is interesting, however, to note that the report for 1907, made to the Commissioner of Public Works by the Genesee Reduction Company, shows that the actual quantity of garbage collected for 1907 was 30 661 tons, or about 18% more than the estimated average for the 5-year period. It is probable, however, that a considerable quantity of garbage was left over by the preceding contractor and included in the 1907 collection.

The estimated population for 1907 was 191 000. The average quantity of garbage per capita per day was 0.88 lb. It is interesting to note that the actual quantity, as shown in Table 2 of Mr. Fetherston's paper, is 0.85 lb.

The Genesee Reduction Company, to which the contract was awarded, acquired a location on the west bank of the Genesee River

between the Upper and Lower Falls. This location is within less than 500 ft. of State Street, one of the important business streets, and within less than ½ mile of the business center of the city. The reduction process used is a modification of the Arnold system, and is called by the contractors the "Beaston process." The plant was put in operation on June 5th, 1907, and has been operated successfully since that time. At first, there were numerous complaints, based very largely on prejudice against the name "garbage plant." These complaints have almost entirely ceased, and the plant has been operated up to the present time without serious offense. Mr. Fisher.

Based upon all the information that the writer has been able to obtain, he would not recommend for the City of Rochester the combined collection of the garbage, refuse and ashes. The ashes and rubbish, at the present time, are collected together, and are used for filling ravines, old quarries, etc. In portions of the city, available dumping places for this mixed refuse are becoming scarce, and the time is at hand when a different method of disposal will be required.

In 1905 the population of the city was 181 670. During the year 253 020 cu. yd. of ashes and rubbish were removed, the estimated weight of which was 116 516 tons. The total cost of this work was $97 208.26. The estimated quantity per 1 000 inhabitants was 1 391 cu. yd., or 642 tons. The cost per cubic yard was $0.384, and per capita, $0.535.

As a result of an examination of 1 208 loads of ashes, rubbish, and paper, during the summer of 1903, it was estimated that 25% of the gross amount was composed of ashes, 28% of paper and 47% of miscellaneous rubbish.

A similar examination, made in January, 1905, of 1 255 loads of ashes and rubbish, showed that 80% was composed of ashes and cinders, and 20% of paper, wood and other miscellaneous refuse. Of the 80% composed of ashes and cinders about 5% was classified as cinders and 75% as ashes. The average for the year was very nearly 50% rubbish and 50% ashes.

The writer believes that his recommendation that the ashes and rubbish be collected separately and the rubbish taken to an incinerator located near the garbage plant, is the best solution of the problem of the disposal of these wastes for the City of Rochester, and he expects the plan will be carried out in the near future. Whether it will be advisable to burn the garbage together with the refuse is a question which the experience of five years with the reduction plant will enable the city to decide better than at present.

The writer again wishes to express his appreciation of the very careful and painstaking work done by Mr. Fetherston, and regrets that the information contained in his paper was not available prior to the making of the writer's report in September, 1906.

Mr. Stearns. FREDERICK L. STEARNS, ASSOC. M. AM. SOC. C. E.—The subject of the disposal of refuse has always been very troublesome. The problem met by Mr. Fetherston in the Borough of Richmond is the same as that found in other cities and towns, and varies only with the different conditions.

The following information relating to the incineration of refuse and the reduction of garbage by the Department of Street Cleaning of New York City may be of interest:

In 1895, when the late Colonel George E. Waring, Commissioner of Street Cleaning, ordered the separate collection of ashes, rubbish, and garbage, it was his intention to incinerate all the rubbish which was not salable. For that purpose, he built quite a complete incinerator plant at East Eighteenth Street, containg a furnace, a conveyor, a boiler, dust catcher, fan exhauster, engine, etc. The refuse was sorted on the conveyor while passing to the furnace, and all salable articles were kept out. Steam, developed in the boiler, furnished all the power required at the plant. An auxiliary furnace was provided for bulky rubbish, such as barrels, boxes, furniture, beds, etc., and such material was not broken up, but was thrown into the furnace as soon as received.

This plant disposed of from 40 to 50 loads per day, and was operated for several years, or until the expiration of the lease of the property.

In the Borough of Queens, the garbage is burned in four Dixon furnaces. These have destroyed 5 617.6 tons of garbage and 687.3 tons of rubbish in one year. In the destruction of the garbage, 626 tons of coal and 12 tons of cord-wood were used. If the wood and rubbish are estimated as equal to 139.8 tons of coal, then 766.1 tons of coal were needed in the destruction of 5 617.6 tons of garbage, or 1 lb. of coal to 7.3 lb. of garbage. These furnaces have been kept in repair, and are still in operation, but the combustion is slow, and the temperatures are not high.

In a 15½-hour test in an experimental garbage furnace built by the Department at West Fifty-first Street, 6 099 lb. of wet garbage were burned by using 720 lb. of coal, or 1 lb. of coal to 8.5 lb. of garbage. The garbage in this case was reduced to 5.6% ash, exclusive of the ash from the coal. No rubbish was burned during this test, as the purpose was to obtain information in reference to burning garbage alone.

A rubbish furnace with two 50-h.p. boilers was next constructed by contract, under John McG. Woodbury, Assoc. Am. Soc. C. E., Commissioner of Street Cleaning. This furnace was designed by H. de B. Parsons, M. Am. Soc. C. E., for incinerating rubbish and experimenting with steam production.

A Decarie furnace, for burning the garbage of the Borough of the

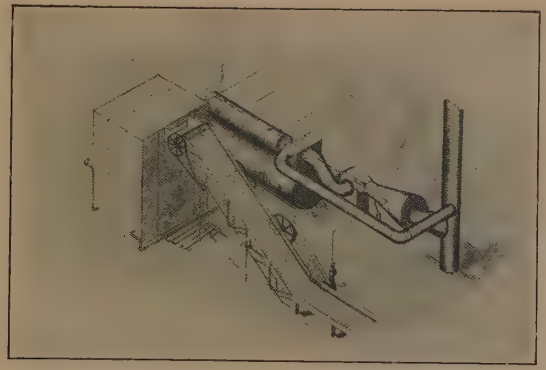


FIG. 1.—INCINERATING PLANT, FOR LIGHT REFUSE, EAST EIGHTEENTH STREET, NEW YORK CITY.

FIG. 2.—INCINERATING PLANT, FOOT OF WEST FORTY-SEVENTH STREET, NEW YORK CITY.

FIG. 3.—SORTING CONVEYOR, WEST FORTY-SEVENTH STREET PLANT, NEW YORK CITY.

FIG. 4.—END VIEW OF INCINERATOR AT FOOT OF DELANCEY STREET, NEW YORK CITY.

Bronx, was built at East 144th Street and operated for several days, but was obliged to close as it failed to do the work. Mr. Stearns.

In a small experimental plant for the destruction of rubbish, constructed by the Department at North Moore and Varick Streets, the following results were obtained:

Duration of test	6	hours.
Quantity of rubbish burned	3 324	lb.
Average horse power per hour developed	22.4	
Quantity of rubbish to produce 1 h-p. per hour	24.6	lb.
Grate surface	40	sq. ft.
Rubbish burned per hour per square foot of grate area	13.8	lb.
Horse power per square foot of grate area	0.56	
Heating surface of boiler	324	sq. ft.
Heating surface per square foot of grate area	8.1	sq. ft.
Water evaporated per pound of rubbish burned	1.5	lb.

Having proved that the rubbish had such a high fuel value, it was decided to alter the West Forty-seventh Street plant and put in an additional boiler of 150 h-p. in order to increase the efficiency of the plant. This work was done by the Department, and, in an experimental test, the following results were obtained:

Duration of test	4½	hours.
Quantity of rubbish burned	23 011	lb.
Average horse power developed	232.7	
Quantity of rubbish to produce 1 h-p. per hour	21.9	lb.
Grate surface	154	sq. ft.
Horse power per hour per square foot of grate area	1.51	
Heating surface of boiler	2 759.9	sq. ft.
Heating surface per square foot of grate area	17.9	sq. ft.
Water evaporated per pound of rubbish	1.59	lb.
Percentage of ash from rubbish	14.5	
Weight of rubbish per cubic yard	111	lb.

There was no sale for the steam power thus produced, and the City had no way of utilizing it.

The following measurements of the light rubbish were made at the West Thirtieth Street dump, the loads being weighed on coal scales, and measured in the carts:

18 loads	19 040 lb.	125¾ cu. yd.
Average load	1 058 "	7 " "

Weighed on small scales, in boxes of known capacity, the following results were obtained:

Mr. Stearns.

Mixed paper, picked.....	5 366 lb.	67½ cu. yd.
Bottles "	272 "	½ " "
Iron "	71 "	¼ " "
Barrels "	1 170 "	13 " "
Total	6 879 lb.	81¼ cu. yd.
Waste	9 382 "	69½ " "
Total disposed of.....	16 261 lb.	150¾ cu. yd.

The decrease in weight, by the second weighing, was 2 779 lb., or 14 per cent.

The increase in bulk, by the second measurement, after sorting, was 25 cu. yd., or 20 per cent.

Percentage picked..... 42% by weight, or 54% by bulk.
" of waste... 58% " " " 46% " "

The following gives the details of another measurement of the light rubbish, made at the West Thirtieth Street dump, the loads being weighed on coal scales, and measured in the carts:

24 loads	21 790 lb.	161¾ cu. yd.
Average load	908 "	6.7 " "

Weighed on small scales, in boxes of known capacity, the following results were obtained:

Mixed paper, picked.....	6 515 lb.	88½ cu. yd.
Bottles "	213 "	½ " "
Iron "	420 "	3 " "
Barrels "	1 446 "	16 " "
Total	8 588 lb.	108 cu. yd.
Waste	10 879 "	92¾ " "
Total disposed of.....	19 467 lb.	200¾ cu. yd.

The decrease in weight, by the second weighing, was 2 323 lb., or 10 per cent.

The increase in bulk, by the second measurement, after sorting, was 39 cu. yd., or 25 per cent.

Percentage picked..... 44% by weight, or 54% by bulk.
" of waste... 56% " " " 46% " "

The following measurements of the light rubbish were made at the West Forty-seventh Street dump, the loads being weighed on coal scales, and measured in the carts:

Mr. Stearns.

44 Department loads.....	48 100 lb.	334¾ cu. yd.
10 private "	4 530 "	39 " "
Total	52 630 lb.	373¾ cu. yd.
Average Department load..	1 093 "	7.6 " "
" private load	453 "	3.9 " "

Weighed on small scales, in boxes of known capacity, the following results were obtained:

Newspapers, picked	5 185 lb.	98 cu. yd.
Manila paper "	1 250 "	54½ " "
Pasteboard "	4 909 "	105 " "
Mixed paper "	2 613 "	53 " "
Rags "	1 007 "	6½ " "
Mixed rags and paper "	625 "	6 " "
Iron and tins "	1 942 "	16 " "
Bagging "	184 "	1 " "
Carpets "	274 "	1½ " "
Barrels "	2 826 "	31 " "
Books "	259 "	½ " "
Bottles "	363 "	½ " "
Shoes "	186 "	½ " "
Hats "	17 "	½ " "
Rope "	111 "	½ " "
Boxes "	1 400 "	11 " "
Total	23 114 lb.	372 cu. yd.
Waste	24 272 "	218½ " "
Total	47 389 lb.	590½ cu. yd.

The decrease in weight, by the second weighing, was 5 241 lb., or 10 per cent.

The increase in bulk, by the second measurement, after sorting, was 216¾ cu. yd., or 58 per cent.

Percentage picked ..	48.8%	by weight, or 63% by bulk.
" of waste..	51.2%	" " " 37% " "

It having been found by actual test that rubbish was a good fuel, it was decided to build a plant under the Williamsburg Bridge, with a conveyor for sorting the rubbish, and with boilers for producing steam.

The plant* was designed by Mr. H. de B. Parsons, and contains two furnaces and two 200-h. p. Sterling boilers, with feed-water heaters.

*A complete description of this plant may be found in *Transactions*, Am. Soc. C. E., Vol. LVII, p. 45.

Mr. Stearns. Very good results were obtained in a 6-hour test of this plant, and were due partly to the furnace being close to the boilers; partly to the use of feed-water heaters; and partly to the longer distance traveled by the heat in passing through the boilers.

In this test the following results were obtained:

Duration of test	6	hours.
Quantity of rubbish burned	40 497	lb.
Average horse power developed	455	
Quantity of rubbish to produce 1 h-p. per hour	14.8	lb.
Grate surface	172¼	sq. ft.
Rubbish burned per hour per square foot of grate area	39.2	lb.
Horse power per square foot of grate area	2.63	
Heating surface of boiler	3 900	sq. ft.
Heating surface per square foot of grate area	22.6	sq. ft.
Water evaporated per pound of rubbish burned	2.29	lb.
Percentage of ash from rubbish	14.9	
Flue temperature between furnace and boiler	2 900°	fahr.
Average weight of rubbish per cubic yard	138	lb.

This plant was used for lighting the Williamsburg Bridge. At first it furnished about 250 amperes at 250 volts, and lighted part of the bridge, but the load was afterward increased until 800 amperes at 250 volts were generated, and this was sufficient for the complete lighting of the bridge. On account of this increased demand for power, the plant was operated beyond its reasonable capacity, and, as a result, the fire-brick lining in the boiler flues was melted. The melted brick, together with ashes and other elements carried from the furnace, filled the flues with a slag resembling iron ore, while the melted bricks gradually disappeared, the tops of the flues caved in, and it was necessary to discontinue lighting the bridge. The plant was then run simply for the incineration of rubbish.

A contract for lighting the bridge was made with the Edison Company for 3½ cents per kw-hr.

In another plant, built by the Department, the boiler is very close to the furnace; there is no flue between the two, the opening between them being the full width of each. This plant has been in operation for nearly a year, and has furnished the power to run the conveyor and the presses, without any repairs, melting of bricks, or production of slag.

It was found, however, that notwithstanding all the improvements made, it would be impossible to generate electricity for 3½ cents per kw-hr., and the project of lighting the bridge in this way was abandoned. The plant has been arranged simply for the incineration of rubbish, and is doing excellent work. The waste heat goes up the stack.

FIG. 1.—INCINERATOR, THIRTY-SEVENTH STREET, BROOKLYN, N. Y.

FIG. 2.—CONVEYOR IN REDUCTION PLANT, BARREN ISLAND, N. Y.

FIG. 3.—CHUTE AND DIGESTERS IN REDUCTION PLANT, BARREN ISLAND, N. Y.

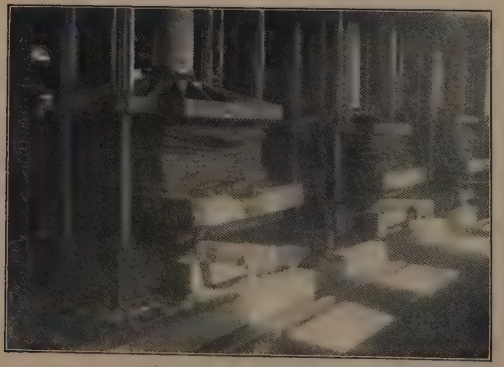


FIG. 4.—HYDRAULIC PRESSES IN REDUCTION PLANT, BARREN ISLAND, N. Y.

Mr. Stearns.

The failure of the incinerating plant to light the Williamsburg Bridge does not prove that rubbish is not good fuel, nor does it prove that it is impossible to generate steam power with rubbish as fuel.

Another furnace (without a boiler) was constructed by the Department at the foot of Thirty-seventh Street, South Brooklyn, and has been found to be very economical. It has been run for nearly two years, with almost no repairs, and the only city employee at the plant is the man who tallies the loads. The plant is operated by a contractor in return for the privilege of sorting out valuable material.

The garbage is subjected to a reduction process by the Sanitary Utilization Company. This company furnishes the scows, tug-boats, and labor required in removing the garbage from the dumping boards along the water front to the reduction plant on Barren Island. For removal from the Borough of Manhattan, the contract price is $148 000 per year, and 187 924 loads were collected and disposed of in 1907, each load weighing about 1 750 lb. The garbage is cooked in digesting tanks until the oil and grease are dislodged, and then the liquids are removed by hydraulic presses. The oil and grease are sold, to be used in the trades, and the solid material from the presses is dried, ground, and used as a fertilizer base.

The following analyses of material were made at the Lederle Laboratory:

June 2d, 1905. Ashes from West Forty-seventh Street incinerator:

Ash	72.68%
Moisture	2.12%
Calorific power	1 173 B. t. u.
Potassium carbonate	2.65%
Phosphoric anhydride	0.91%

June 9th, 1905. Ashes from the West Forty-seventh Street incinerator:

Ash	54.71%
Moisture	4.01%
Moisture in original (approximate)	50%
Calorific power	4 682 B. t. u.

June 23d, 1905. Street sweepings from Rivington Street, Lower East Side, Manhattan Borough:

Moisture (approximate) in sample	66%
Moisture, in dried and powdered sample	0.49%
Ash, in dried and powdered sample	37.56%
Calorific power	6 068 B. t. u.

Mr. Stearns. June 23d, 1905. Street sweepings from Fifth Avenue, between Twenty-third and Fifty-ninth Streets:

Moisture (approximate) in sample as received	60%
Moisture, in sample as analyzed	0.64%
Ash, in sample as analyzed	14.85%
Calorific power	8 600 B. t. u.

August 13th, 1905. Garbage from Pitt Street, between Rivington and Stanton Streets; and from Rivington Street, between Pitt and Sheriff Streets:

Moisture, in original	60%
Moisture in material as used for calorimeter test	1.87%
Ash in material as used for calorimeter test	17.40%
Calorific power	8 799 B. t. u.

November 10th, 1905. Ashes from Delancy Street incinerator:

Ash, in dried sample	79.85%
Combustible matter, in dried sample	20.15%

Sample was wet when received, evidently from sprinkling.

January 18th, 1906. Ashes from Delancey Street incinerator:

Nails and other metal	5.48%
Broken glass	4.05%
Bone phosphate	2.71%
Potash	0.46%
Alkaline earth carbonates, silicates, oxides of iron and alumina, etc	60.91%
Moisture	0.75%
Organic and volatile matter (loss on ignition)	25.64%

January 18th, 1906. Ashes from West Forty-seventh Street incinerator:

Moisture	2.12%
Potassium carbonate	2.65%
Calcium phosphate	1.98%
Alkaline earth carbonates, silicates, soda, oxide of iron and alumina, etc	68.05%
Organic and volatile matter (loss on ignition)	25.20%

Mr. Morse. WILLIAM F. MORSE, ESQ. (by letter).—The conditions attending the collection and disposal of the municipal refuse of New Brighton, as described by Mr. Fetherston, are similar to those in the great ma-

jority of American towns of the third and fourth classes. The surveys and reports he has made may apply equally well to many other places, and, by comparison, will be of the greatest value to those interested in this subject. He has laid a firm foundation upon which all may build. Mr. Morse.

The quantities and seasonal variation in Table 2, do not correspond with the observations contained in several reports from other towns. The weights per cubic yard for ashes and rubbish, 0.42 ton, and garbage, 0.46 ton, are much lower figures than have formerly been accepted as standard.

This difference is undoubtedly due to the mixed collections, but, making allowances according to the subsequent determinations for rubbish and deducting these, it will still appear that his average weights per cubic yard of separated garbage and ashes are much below those reported from other places.

The calorific values of the several classes of waste, determined so thoroughly by the calorimeter and analysis of Mr. Welton in Table 6, and supplemented by the practical tests covering the period of a year, are the most convincing proof of the combustible character of American city refuse.

There have been many statements to the effect that American refuse could not be burned as is done abroad, even with the same forms of destructors, because of the larger relative quantities of wet garbage, carrying greater quantities of moisture, and with household ashes containing 50% of fine ash—worthless for fuel.

To those engineers whose work is connected directly with the disposal of waste, this part of the paper is perhaps of the greatest interest. It answers many questions which heretofore have remained unsolved; determines the calorific value at different periods; proves the deductions by practical tests, scientific demonstrations and analyses; and gives a standard for the measurement of all work done by any form of crematory furnace.

The two sets of calorimeter tests, made by different authorities, and covering a period of two years, show the thoroughness of the work. The first set of tests shows an average of 3 127 B. t. u. for the year, while the tests in the second year show 4 274 B. t. u., the difference being probably due to better care in the selection and treatment of the material. The average of these determinations raises the heat values of American refuse to a place quite equal to the English standards, while the equivalent evaporation, 2.03 lb. of water per pound of refuse, as shown in Table 9, seems to furnish conclusive proof that results equal to the English work can be obtained from the combustion of American waste in properly designed furnaces.

To make this clearer, the writer has ventured to extend the data tabulated in the first part of Mr. Fetherston's paper, and express this in equivalent quantities of coal.

Mr. Morse. TABLE 14.—THEORETICAL CALORIFIC VALUES OF AMERICAN CITY WASTE, IN EQUIVALENT COAL.

Combined waste:	1 ton ashes, garbage, refuse, and rubbish	480 lb. coal.
	1 " garbage, refuse, and rubbish	502 " "
	1 " ashes, refuse, and rubbish	532 " "
Separated waste:	1 " ashes	487 " "
	1 " garbage	363 " "
	1 " refuse	1 298 " "
	1 " rubbish	

In this statement, refuse is taken to mean dry combustible matter—paper, wood, straw, rags, etc., everything except ashes, garbage, and rubbish. Rubbish means incombustible matters—metals, glass, crockery, etc.,—and is a subdivision of refuse.

In examining Table 8, which reports the practical tests in burning the waste of New Brighton as actually collected during a period of one year, the question of temperatures seems to be worth some consideration. The apparatus used was not well adapted for accurate work, and there could be no reliable records except for a part of the year (March to September, 1906). Taking the last fourteen trials, when the temperatures were recorded by pyrometer, the highest is 1 700°, and the lowest 1 100°, the mean being 1 380° fahr. These measurements were taken directly behind the fire-box of a Dixon crematory, with which the tests were made, and are not comparable with the temperatures obtained in the combustion chamber of a modern destructor; but, as far as they go, they are the only set of temperatures recorded for American crematory work, when under forced blast, and are the more significant for comparison with the usual operation of crematories by natural draft.

The writer was present at one of these practical tests (September 26th, 1906) when 1 200 lb. of mixed waste was destroyed. The proportions were approximately: Garbage 60%, ashes 30%, refuse 7%, rubbish 3%. The time was 1 hour and 10 min. The conditions and apparatus were as described in the paper, and the temperatures behind the coal grate were from 900° to 1 400°, average 1 035° fahr. As the purpose was to ascertain the approximate calorific value remaining in the clinker, no withdrawal of clinker or ashes was made until the whole quantity of waste had been burned. The clinker was withdrawn as quickly as possible, and a blast of cold air at a pressure of 1¼ in. of water was passed through this, and the readings for temperatures taken at intervals of 1 min. The first temperature was 423°, and in 6 min. it fell to 150° fahr.—a loss of 273 degrees. The weight of the clinker was 272 lb., and of ashes 73 lb.—the whole being 345 lb., or 28% of the original quantity destroyed. There could be no doubt of the perfect combustion of mixed waste without other fuel under cold-

air blast, even with the great loss of heat due to imperfect appliances Mr. Morse.
It also shows that these results could not have been obtained by the natural draft in a chimney of reasonable height with a crematory of this form.

A subsequent determination of the temperatures in this crematory, while in actual work, burning the daily collection of garbage, using coal as fuel, was thus reported:

Dixon crematory, 25 ft. long, four charging holes, four stoking doors, heavy coal fire, with usual charge of city garbage and light refuse. Temperatures: immediately behind firebox, 2 000°; at first stoking door, 1 500°; at second stoking door, 1 160°; at third stoking door, 970°; in chimney, 14 ft. above furnace, 606 degrees. The loss of heat, from the fire-box to the base of the chimney, was approximately 1 400°, a loss of about 350° for each 5 ft. in length of furnace garbage grate. There was no re-heating or re-burning of the gases, which were discharged from the stack in heavy, dark clouds.

In the summary of the work of the English destructors, the author gives the net cost for labor at 21½ cents per short ton, increased for American labor conditions to 43 cents per ton.

When destructor plants are operated for power, especially for electric lighting, the expenses are slightly increased, because of the need of power development within a limited time.

In relation to this, it will be of interest to include an abstract of the report of the Meldrum destructor at Westmount (Montreal), which has been in service almost two years—long enough to obtain some data of value. (See description of plant, Table 10, No. 40.)

The quantities and seasonal variations in the composition of the Westmount waste are approximately as follows:

Quantity, 1906— 8 months—more than 3 000 tons.
" 1907—12 months—about 8 000 "

Composition:

Items.	Summer.	Winter.	
Garbage	60%	20%	
Ashes	20%	70%	(including much fine dust.)
Refuse	20%	10%	

Average daily quantity destroyed, summer, about 15 to 20 tons.
" " " " winter, " 30 to 40 "

Estimated coal equivalent per ton of waste (average) (coal cost $5.00 per ton)	=	$0.58
Total return in cash credited to destructor, 1906	=	3 090.00
Total return in cash credited to destructor, 1907	=	4 636.00
Total net operating costs and fixed charges, 1907, including interest 4%, deprecia-		

Mr. Morse.

tion 4% and sinking fund 1% (after crediting sale of steam to electrical plant)	=	$6 055.00
Total net operating costs and fixed charges per ton for 1907, after crediting sale of steam	=	0.75
Total net operating costs for 1907, after crediting sale of steam	=	2 423.00
Total net operating costs per ton for 1907, after crediting sale of steam	=	0.30
Average temperature in combustion chamber.	=	1 500° to 2 000° fahr.
Hours of operation, summer		7 A. M. to 7 P. M.
" " " winter		7 A. M. " 7 A. M.

As there is a very large percentage of absolutely valueless fine dust-like ash mixed with this refuse, especially in winter, due to the large number of sifting furnaces installed in Westmount houses, and also because of the much higher rates of wages paid for operators, the cost per ton is higher than the average figures from English destructor service, but, with the fine ash screened out (as is now contemplated), much larger quantities of refuse can be handled, and far better results obtained; the costs of operation per ton could also be much reduced if the refuse were forced through the destructor furnaces as fast as it would burn, instead of being burned on the grates comparatively slowly, so as to get the best steam results from it.

With the conclusions and deductions formulated by Mr. Fetherston, all engineers who have studied this subject must, in the main, concur. While there may be differences of opinion as to the adaptation of these forms of destructors for the general work of waste disposal in America, there certainly can be but one conclusion when the conditions and surroundings are such as are reported in this paper.

If the results are as expected, and the work of waste disposal in New Brighton is successfully done, a long step will have been made in the direction of a satisfactory solution of a puzzling municipal problem.

Mr. Newton. E. B. B. NEWTON,* ESQ. (by letter).—This able paper contains a vast amount of information upon the subject of municipal refuse, and it is arranged in a systematic and scientific way. Apparently, it considers minutely only one aspect of the question, namely, that of the disposal of refuse by fire, and little reference is made to other systems of disposal, which, under certain circumstances, may be right and proper ones to use, even in a large city or town.

Theoretically, the destruction of such refuse by fire is not commendable, as it necessarily involves the waste of certain materials, and the object to be sought seems to be a system which will enable

*Borough Surveyor, Paddington, London, England.

this waste material to be put to profitable use for the benefit of the community. Mr. Newton.

Among other systems of disposal, the following may be cited:

(1).—The utilization of such refuse for the purpose of filling up low-lying lands. In the writer's experience, this system has been adopted in the case of an English city where a desolate area of land, under water for many weeks of the year, was turned into a well-drained and attractive public park, the level having been raised in some places many feet, while the refuse of the town was disposed of at a trifling cost.

(2).—The distillation of such refuse, the gas evolved being utilized commercially.

(3).—The conversion of the whole of the refuse into a fine powder, which, mixed with chemicals, may be advantageously used as a manure.

(4).—The separation of the refuse by machinery, or by hand, into its constituent parts, whereupon, in the first place, the materials may be disposed of more cheaply when all of one kind are together; and, secondly, the materials thus separated may be utilized for commercial undertakings.

In the Borough of Paddington, with which at the present time the writer is connected, the last system is now in use, because it is the most advantageous and economical under the particular circumstances.

An estimate* of the comparative costs of house refuse collection and disposal in Paddington with and without the sorting of the refuse in the manner described, and based upon the actual expenditure for the year 1900-01, may be roughly summarized as follows:

	£.	s.	d.
Cost of removing in barges 20 000 tons of refuse, at 3s. 8d.	3 666	13	4
Whereas there is obtained, by sifting and selling	1 160	3	2
Making a total gain, by sifting, of	4 826	16	6
Less the cost of working the sifting machine	2 163	12	0
Balance in favor of sifting	£2 663	4	6

The ashes and breeze which form the principal source of income, are used in the brickmaking industry, being mixed with the clay of the London Area, formed into bricks which are then burnt in clams, to produce the ordinary London stock brick.

It is true that the total amount of the sales depends considerably upon the state of the brickmaking industry, and the price ob-

* Mr. Newton's discussion was accompanied by an estimate, in considerable detail, of the comparative costs of refuse collection and disposal with and without sorting. This estimate is filed in the Library of the Society, where it may be examined by anyone interested.

Mr. Newton. tained for ashes and breeze is generally regulated by the state of the market at the time of production. The bricks are usually sold at a rate per chaldron. On occasions this rate has been as high as 5 shillings, while, within the writer's recollection, as much as 3 pence per chaldron has had to be paid for the removal of the ashes and breeze. Even when this amount is paid, the system is economical, for it is cheaper to dispose of the various materials when sorted into their several kinds than to endeavor to get rid of them in a rough miscellaneous mass.

In past years, destructors have been proposed, but there is no reason at present to regard them as likely to be more economical or less offensive than the present system, while the objections of the residents of a high-class district in the west end of London to a destructor, whether well founded or not, are very numerous, and very powerful.

Mr. Hering. RUDOLPH HERING, M. AM. SOC. C. E. (by letter).—Mr. Fetherston's paper has supplied information concerning the disposal of municipal refuse which has been needed in the United States for years, and furnishes facts that are fundamental to the proper treatment of the subject. The writer, as Chairman of the Garbage Committee of the American Public Health Association, urged American cities, as far back as 1890, to obtain such information through the co-operation of engineers. Again, in a paper before the International Engineering Congress at St. Louis, in 1904, the writer urged the doing of experimental work, and called attention to the special questions to be solved. Not until the subject came under the immediate direction of engineers has it become possible to make substantial progress.

The reason why health departments were entrusted with the treatment of this subject was chiefly the offensiveness of the garbage. The real sanitary danger, however, does not lie in the decomposition of domestic garbage, which rarely contains disease germs, but in the rubbish, which is made up of dust, sweepings and discarded refuse, including that of sick rooms, such as mattresses, rags, etc. While sanitary authorities should have a certain amount of jurisdiction in this matter, the practical methods of securing an economical disposal must fall to the engineer.

To the writer's knowledge, there is but one other instance where a similar scientific and thorough investigation of the refuse disposal question has been made. It likewise extended over a year, from 1895 to 1897, taking in seasonal variations, and was made by two engineers, Messrs. Bohm and Grohn, to determine the practicability of burning the refuse of the City of Berlin. The study was similar to that made in the Borough of Richmond, and in some respects it was on a larger scale. It proved that incineration of the Berlin refuse, due to the absence of a sufficient quantity of unburned coal in the ashes, was not economical under the conditions then existing.

Mr. Hering

Mr. Fetherston and the Commissioner of Public Works of the Borough of Richmond deserve credit for bringing the question of refuse incineration into the realm of engineering, where it unquestionably belongs. The supply of more exact information regarding the composition of refuse materials, their quantities per inhabitant, their seasonal variation, and their calorific power will be of benefit, not only to the Borough of Richmond, but to many other cities of a similar character.

The remaining part of the whole problem, the results of the actual incineration of large quantities of refuse, throughout all seasons, in a modern well-designed furnace, will, after one or two years of operation, place American engineers for the first time in a position to deal scientifically with the subject and to assure satisfactory results at the least practicable cost.

There are a few points which the writer desires to emphasize, believing that upon them further light is needed, and that this, no doubt, will soon be supplied after refuse incineration in American cities has been fully inaugurated.

Thus far, the English incinerators have practically given the best information as to results, although such results are almost entirely empirical, and sometimes even crude and indefinite. It is not assured, therefore, that the precise results obtained in England will be repeated in America, because American engineers are not yet in a position to forecast fully, on a large scale, the effect of the differences in the character of European and American refuse.

The use of bituminous coal in England and of anthracite coal in most cities in America is one of the conditions which will affect the result materially. Anthracite coal yields a much larger amount of ash, much of which is fine dust, but bituminous coal produces clinker. In Montreal, with a mixed refuse collection, it was found necessary, after a few years of operation, to screen the refuse, and eliminate the fine ash, in order to have a satisfactory combustion, as the fine ash naturally prevents air from passing through the mass as freely as through the clinker. In the adjoining city of Westmount such screening does not appear as yet to be necessary. More experience is required to prove on a large scale the measure of this difference, and whether a prior screening is economical or otherwise.

There is a further difference in the refuse material of the two continents. It has been stated that the mixed refuse in England contains more moisture than the mixed refuse in the United States. If this is a fact, it is likely that it is due to the greater humidity of the air in England, and therefore the moisture may be found to be largely in the fine pores of the ash. Moisture in ashes in humid climates shows itself as a thin film of water which surrounds each particle and which capillarity brings to the surface if there is evaporation at the

Mr. Hering. surface. Moisture in the organic matter of summer garbage (melons, corn-cobs, etc.) is contained in almost closed cells, not in capillary channels as found in ashes. When moisture is heated and passes as steam through garbage, it is decomposed by incandescent carbon, the hydrogen being liberated and the oxygen burning the carbon of the garbage, which produces a high temperature. Steam is created more readily from the moisture contained in the open capillary channels of incompletely burnt ashes by the incandescence of the remaining carbon, than from the moisture confined in the cellular masses of organic matter, which not only have less heat-conducting power, but from which moisture is not as readily drawn out and turned into steam by incandescent carbon as from the capillary spaces in ashes. Therefore, free capillary moisture in ashes should not reduce the temperature as much as the confined moisture in meats and vegetables; or, in other words, the reported moist mixed refuse of England, but with less garbage, may burn better than the drier refuse of cities in the United States, but containing more garbage. It is the writer's opinion that the precise effect of the moisture in the melon season still needs the demonstration that it will get on a large scale in a well-designed furnace such as has recently been built at New Brighton.

Still another question should be answered more satisfactorily than has been done, up to the present time. Will a mixed collection be profitable in the largest cities of America if the refuse is to be incinerated? Unquestionably, the popular desire favors a mixed collection. It was difficult to introduce a separation of garbage, ashes, and rubbish, and there still exists everywhere a feeling against the slight additional trouble which it causes, while it has been easy to abandon separation and return to the one-barrel collection; but, if there are material advantages in the separation, for the collection and disposal of the entire refuse, such slight additional trouble should not be a factor.

A mixing of garbage and ashes will tend to draw out, by the capillarity in the latter, some of the moisture contained in the former, and allow a greater evaporation to take place; but, with the kind of garbage in America, where about 10% of the contained moisture will run out of it by the pressure of its own weight (more or less according to the season), it needs a further demonstration to determine whether it is advisable to have a mixing at the house or not until after a delivery at the furnace, and after an opportunity has been given in one way or another to free the garbage economically of its excess of moisture. It is much better that moisture, from the material to be burned, be removed outside than inside of the furnace, if other conditions are equal.

In view of the greater quantity of fine ash in anthracite coal, the conditions when a separate disposal of ashes is cheaper will occur more frequently than when bituminous coal is used.

Mr. Hering.

Another important question relates to the maintenance of a high temperature, both for sanitary and steam-raising purposes. As the refuse has no high fuel value, when compared with coal, it is necessary to make special effort to economize all the available heat. A water-jacketed furnace, therefore, is quite objectionable, also every contrivance and method of operation which will allow cold air to enter the furnace and reduce the temperature. On the other hand, every means should be utilized to conserve the available heat. All air used for forced draft should be heated by the hot gases when about to escape through the chimney, and the heat of the ashes after clinkering should also be utilized.

It is well known that recent methods of garbage incineration in America have been costly and unsatisfactory in their results. With the greater knowledge of all conditions bearing upon the problem, which has been augmented so efficiently by the work done in the Borough of Richmond, undoubtedly better, and the writer believes quite satisfactory, results will be obtained at no greater cost, and in many instances at a much smaller cost than heretofore recorded.

To reduce the cost of incineration, it will be necessary to utilize for revenue as fully as practicable the heat which is produced by the combustion. There appear to be no inherent difficulties in the way of doing this. Each local problem must be considered in its own way, and the question of heat utilization should be a part of the entire problem. Continuous incineration, giving continuous power, could be used for pumping water or sewage, or for electric traction. Intermittent incineration could be used for electric lighting or for manufacturing purposes.

In order to accomplish satisfactory results, both in maintaining high temperatures and in supplying a steady power, it is highly important to have trained men under skillful mangement, and this requirement will naturally make slow progress at first; but it is to be hoped that the New Brighton furnace, which starts under such excellent guidance, will clear the way to make this progress not only effective, but more rapid than could otherwise be expected.

When it is assured that high-grade incinerators are practicable, and that trained men are available, then it will also become practicable to place such incinerators within the built-up parts of a city, without any objectionable results. The nearness of the furnace to the source of the refuse will be a means of economy by shortening the haul and reducing the cost of collection. This greater economy secured for collection will permit, if necessary, a larger margin of the expense to be applied to a better operation of the plant; and the nearness of the plant to the center of industrial activity will give even more opportunities for utilizing the power developed.

Mr. Fetherston.

J. T. FETHERSTON, ASSOC. M. AM. SOC. C. E. (by letter).—In closing the discussion on this paper, it may be stated that due consideration was given all known methods of refuse disposal before mixed refuse destruction was selected as the most suitable for the conditions in Richmond Borough; garbage cremation, in particular, received thorough investigation. In 1895, Theodor S. Oxholm, M. Am. Soc. C. E., Engineer of the Village of New Brighton, now a portion of the Borough of Richmond, examined and reported on furnaces at Norfolk, Va., Wilmington, Del., Scranton, Pa., and Terre Haute, Ind. Mr. Richard T. Fox, Superintendent of Street Cleaning in Richmond from 1902 to 1904, visited a number of incinerators, and made a special report on the plant at Minneapolis, Minn. The writer spent a week during the month of August, examining and testing the crematory at Atlantic City, N. J. Furnaces in the vicinity of New York City, including one in the Borough of The Bronx, those in Queens Borough, at Coney Island, and at the Long Beach Hotel, were also inspected.

Mr. Venable's valuable book on "Garbage Crematories in America" had not been published when installations for Richmond Borough were under consideration, but the main features of most of the American crematories were known to those in charge of refuse disposal through articles in the engineering press, by means of pamphlets and by visits from inventors and builders of furnaces.

Two types of garbage crematories had been operated long enough in Richmond to disclose the weaknesses in their design and construction, and none of the furnaces visited promised to overcome successfully the inherent defects of the local crematories so as to prove permanently sanitary and economical in operation.

Reduction was not considered feasible on account of the small quantity of garbage contributed by 60 000 people scattered over a district about 8 miles long by 2 miles wide, and because a suitable location for such a plant would render the cost of collection and removal prohibitive.

After the failure to secure economical transportation by street railway to an outlying property, the destruction of mixed refuse seemed to be the only method in successful operation which promised a satisfactory solution of the local problem.

To reduce the water content of garbage by pressing or squeezing out the moisture before burning the material seems to be a favorite suggestion when the incineration of garbage is under discussion. A trial of this method was made one hot day in August, 1904, at the West New Brighton crematory. Two steel rolls, pressed together by springs, carried a horizontal feeding apron held in position by a third roller. Power for operating the machine was furnished by a belt-connected electric motor. When the trial began, there were a few

flies buzzing about the room, but within 5 min. after garbage was started through the press, the place literally swarmed with flies; evidently the good news or the delicate (?) odor reached far and wide; flies clung to the slimy, dripping rolls, and refused to let go; some were squeezed to death either because they were stuck fast or preferred to risk their lives rather than lose the feast. As a result of this experiment, which caused obnoxious smells about the plant and only macerated the garbage which still clung to the rolls, the writer became convinced that the squeezing of water from raw garbage by mechanical means was inadvisable, at least under the conditions of the trial. If mechanical treatment of garbage should appear necessary or desirable, it would seem better to adopt the full reduction process, where some by-products would be obtained to offset the cost of the method. Mr. Fetherston.

No incinerator of the type suggested by Mr. Venable, in which garbage, ashes and rubbish would be burned separately, is known to the writer. Such estimates, data, and actual trials as have been made for conditions in Richmond Borough lead to the conclusion that mixed refuse destruction will prove not only more sanitary than separate disposal of ashes, garbage, and rubbish, but may also prove more economical in the collection and disposal of the wastes. Certainly the householder will be pleased to have only one receptacle instead of two, and the garbage-can, which is such a source of annoyance, particularly in hot weather, may be discarded. Garbage will be mixed with ashes, and decomposition will be arrested, to a great extent, by the absorption of moisture in the fine ash. Collection wagons for the removal of mixed refuse should be much less offensive while on the streets than garbage carts. All the advantages in collection and disposal seemed to favor mixed refuse destruction in the problem under consideration. For other cities, however, the conditions undoubtedly will vary, and each particular case must be considered on its own merits as an engineering problem. It is believed that there are hardly sufficient reliable data available at the present time regarding sanitary efficiencies and costs of collection and disposal of refuse upon which to base broad conclusions. Generalizations founded upon inadequate premises are more apt to do harm than good in bettering present methods.

It will be of interest and value to have the method of gas analysis suggested by Mr. Cary compared with calorimeter tests of refuse in an actual trial of a destructor. A complete gas analysis, with samples taken at frequent intervals during the test, may prove satisfactory in computing the calorific power of the material burned, though whether an exact heat balance is possible may well be doubted. Mr. Welton's comparisons of the variations in fuel values of the combustible in coal and refuse is suggestive in this respect, and his summary in Table 12 indicates that the calorific power of refuse may be determined by calorimeter with almost the same degree of accuracy as is

Mr. Fetherston. obtained in tests of coal. The chemical analyses, and the computed heat values therefrom, further strengthen this contention, and provide additional information concerning the composition of refuse. The details of the laboratory tests, brought out in Mr. Welton's discussion, serve a useful purpose in supplementing and amplifying the statements in the paper, and provide information which defines the limits in the application of the experimental data.

A reasonably high temperature is necessary in burning refuse, so that the formation and escape of obnoxious gases may be prevented. The writer certainly did not intend to fix the minimum desirable temperature of the products of combustion at 1 250° fahr., as noted in Mr. Cary's discussion. The limit below which the temperature of the gases in the combustion chamber of a furnace should not fall was placed at 1 250° fahr. Mr. Foster has shown that higher temperatures than 2 000° fahr. are undesirable, and the writer's observations confirm his statements.

It might be possible to combine the suggestions of Mr. Green and Mr. Foster in utilizing the power from refuse for heating houses in winter and making ice in summer. Municipalities in the United States, however, have not yet approved of municipal trading to this extent in competition with private ownership. Many uses can be made of the steam power resulting from the burning of refuse, but legal restrictions and public opinion may prevent officials from selling the by-products to the best advantage. The power from refuse might well be utilized in decreasing the cost of collecting the wastes, by using electric trucks driven by storage batteries charged at the destructor, provided this proves economical. This proposed method of collection would not interfere with private ownership, nor would it be hampered by legal restrictions. The possible decrease in the cost of refuse destruction by the utilization of the power produced will undoubtedly be determined by local conditions in any case.

Where the reduction system is in operation, among the larger cities of the United States, Mr. Koyl's proposal to recover the coal found in ashes is certainly worthy of attention, and it is hoped that a machine for this purpose may be constructed and operated, in order that the economy of the process can be determined.

Mr. Tribus has called attention to the transmission of disease by the common house fly—"The fly that doesn't wipe its feet." It seems odd that this insect, which breeds in filth, preferably horse manure, and carries all sorts of germs about on its legs and body, should have been neglected so long, because its power for conveying disease is so evident. It is quite possible in the future that American methods of refuse disposal will be greatly influenced by further investigations concerning flies and the transmission of disease.

Attention is called by Mr. Cole to a private plant in Chicago producing power and revenue from selected wastes similar to rubbish.

Incinerators for burning municipal rubbish are operated in New York, Boston, Buffalo and Brooklyn. At Delancey Street, Borough of Manhattan, power resulting from the burning of rubbish was used for a time in lighting the Williamsburg Bridge, but this use, the writer understands, was discontinued a short time ago. It is hardly believed that such installations as the one mentioned by Mr. Cole will interfere seriously with furnaces burning mixed refuse, as manufacturing waste of the character noted is not usually removed and disposed of at public expense. Mr. Fetherston.

The valuable discussion by Mr. Hering considers among other factors the effect of fine anthracite ash on the destruction by fire of mixed wastes, and this matter will receive attention in the practical operation of the West New Brighton destructor. Mr. Hering also considers that the removal of moisture from garbage would better be done outside of the furnace, if other conditions are equal. It is questionable, however, as to whether it is more sanitary and economical to try to remove moisture by special devices than to evaporate the water in the furnace.

It does not appear to the writer that the presence of moisture in summer garbage or the moisture carried by steam jets used for forced draft can by any means help the temperature of a furnace, as suggested by Mr. Hering. The presence of moisture in any form tends to lower temperatures. The dissociation of water into the so-called water gas is initially a cooling process, and whatever gain may be obtained, by the recombination of the hydrogen and oxygen later in the process, will hardly result in a net gain in furnace temperature. This matter of the formation of water gas has been the subject of much discussion in Great Britain in connection with refuse destruction.

The problem of refuse disposal is wholly one of engineering, and not a matter to be handled by boards of health or city councils. This view has been held by Mr. Hering for a number of years, and is undoubtedly in a fair way to be realized.

Analysis of the faults in the design of garbage crematories by Mr. Leask should prove of value to engineers interested in the construction of furnaces. Undoubtedly, many of the American crematories were crude in design; however, the cremation of garbage alone is a very different problem from the burning of mixed refuse, and allowances should be made when principles of design are compared.

Mr. Leask's criticism of the figures in Table 1 may well be applied to a good portion of the published information concerning the composition of refuse. In but few cases are the limits of the data defined, or statements made of the basis upon which the information was obtained. Empirical formulas are often over-worked in engineering computations, and the components of refuse in many cases have been manhandled out of all resemblance to the original material, in the attempt to make much out of little. The data from Torquay in

Mr. Fetherston. Table 10 would appear to be inverted, as suggested by Mr. Leask, but the figures agree with those quoted by Mr. Goodrich in his book on the "Disposal of Towns Refuse."

As a rule, writers on city waste disposal have been of the opinion that American refuse contains more moisture than British refuse. The writer's opinion, based on personal examinations of refuse at the destructors noted in Table 10, was contrary to this belief, when British material was compared with the refuse of Richmond Borough. Mr. Leask's corroboration of the writer's statements adds weight to the opinion expressed, though a distinction has been noted regarding the character of the moisture. British refuse contains water due to a moist climate, while most of the moisture in local refuse is contained in the garbage. It is of further interest in this regard to notice the large proportion of garbage in the analysis of refuse at Stoke Newington and Kings Norton, England. If reliable comparisons could be made between the composition of refuse from different cities, suitable methods of disposal would be more certain of success, though, as Mr. Leask has pointed out, "such data must be used with the discrimination born of experience."

The writer appreciates the position in which Mr. Fisher, of Rochester, found himself after reviewing the available data on refuse disposal, and the determination to adopt the reduction system would seem to have been the wisest course under the circumstances.

Engineers who are interested in the burning of rubbish will find Mr. Stearn's discussion most significant, and the figures submitted should greatly assist in a better understanding of the difficulties to be surmounted in the disposal by fire of this class of waste.

Paddington is rather an exception to the general practice in Great Britain, of destroying refuse by fire. Apparently, local peculiarities govern in this case, and Mr. Newton's contribution is of more than ordinary interest in proving that there is no hard-and-fast, best method of refuse disposal applicable to all conditions.

The cost figures for the Westmount destructor would prove of greater comparative value if given in more detail by Colonel Morse, as rates of wages vary in different localities, and the apportionment of credits in combined destructor and electrical power installations (as is the case at Westmount) is often based upon arbitrary agreements rather than actual benefits derived from burning the refuse. The net operating costs and the fixed charges at Westmount would seem to be much lower than for Stoke-upon-Trent, England, where wages are only about one-half, and where the destructor furnishes all the power for the adjoining electrical station.

The writer is pleased with the interest made evident by the valuable discussions on the paper, and hopes that results of practical import in the destruction of refuse by fire may be developed from the data submitted.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1075

THE REINFORCED CONCRETE WORK OF THE McGRAW BUILDING.*

By William H. Burr, M. Am. Soc. C. E.

With Discussion by Messrs. H. F. Tucker, W. J. Douglas, Carl Gayler, J. A. Jamieson, Walter M. Smith, Clarence W. Noble, Guy B. Waite, E. P. Goodrich, T. L. Condron and F. F. Sinks, E. W. Stern, L. J. Mensch, P. E. Stevens, and William H. Burr.

The McGraw Building† is a true reinforced concrete structure—the latest type of buildings of that general class. It is on the north side of West Thirty-ninth Street, between Seventh and Eighth Avenues, in the City of New York, in that district which has already felt the stimulating influence of the new Pennsylvania Railroad Station in process of construction a half dozen blocks to the south. This part of the city is undoubtedly destined to become a great business center, where substantial buildings of the highest type will be required in order to meet the demands of the development of that vicinity.

The building has a frontage of 126.3 ft. on Thirty-ninth Street, and a depth of 90 ft. It has eleven stories, and a penthouse, or roof structure, nearly equivalent to another floor. The height of the roof is 145 ft. above the ground floor, or nearly 150 ft. above the street, or, finally, 159 ft. 6 in. above the basement floor. While, therefore, it is

* Presented at the meeting of November 20th, 1907.

† Messrs. Radcliffe and Kelly were the architects of this building, with whom the writer was associated in the work covered by this paper.

far from ranking among the tallest sky-scrapers of the city, it is to be classed among the high business buildings of Manhattan Island. Its height is much greater than has heretofore been considered practicable for a purely reinforced concrete building, *i. e.,* a concrete building without iron or steel columns.

It has been constructed for the McGraw Realty Company primarily to accommodate the business of the McGraw Publishing Company, whose publications include *The Engineering Record, Electrical World, Street Railway Journal* and the *Electro-Chemical* and *Metallurgical Industries.* At the same time, it was designed to accommodate not only the printing presses of the McGraw Publishing Company, but any other similar business requiring the operation of heavy machinery or the storage of heavy goods in bulk. It was imperatively necessary, therefore, that the building should be designed and built so as to afford the greatest possible resistance to the vibration of heavy machinery, and possess to an unusual degree both rigidity and durability. It is also fire-proof to such an extent that the McGraw Realty Company may reasonably be its own insurer. While the building is admirably adapted to office use, its lower floors, particularly, are thus capable of affording provision for those business purposes which require heavy and substantial construction.

Like most other portions of that part of the city north of Fourteenth Street, the rock originally at the site of the building was close to the surface. The excavations for the foundation were not carried deeper than about 20 ft. below the street surface, and there the entire foundation was placed upon the gneiss which forms the ledge or bed-rock. There were no real foundation problems to be solved. The columns supporting the building, and the retaining or area walls around the basement, were all founded upon the same ledge, under the requirements of the Building Code of the City of New York.

In order to meet the exacting requirements for the unusually substantial structure required by the McGraw Publishing Company, it was decided to design the building for a live load of 250 lb. per sq. ft. for the first and second floors, 200 lb. per sq. ft. for the third floor, 150 lb. per sq. ft. for the fourth floor and 125 lb. per sq. ft. for all the other floors above the fourth, and with a live load of 60 lb. per sq. ft. for the roof. All parts of the floor beams and girders, therefore, and the columns, were designed to sustain, under the requirements of the

THE MCGRAW BUILDING.

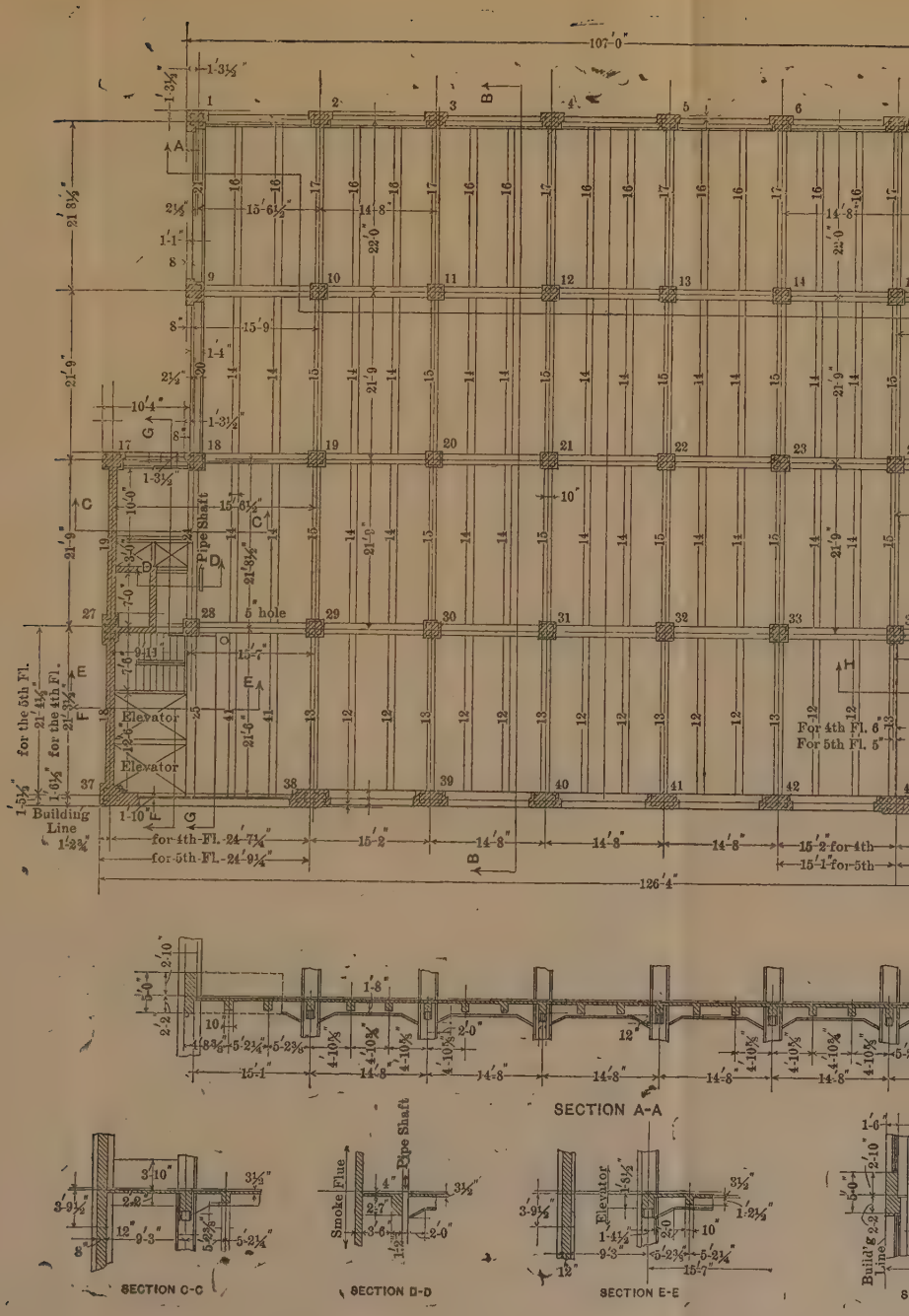
Building Code, the actual weight of the structure and the live loads specified above.

Prior to the submission of this design, the Building Code of the City of New York had permitted a working stress of only 350 lb. per sq. in., for concrete in direct compression, it being practically assumed that all reinforced concrete columns would be of the Considère type. Obviously, if this regulation should prevail for an eleven-story building, the size of the columns in the basement and lower stories would be so great as to trench too seriously upon the available space for machinery or for other business purposes. As it was strongly desired to secure all the material advantages accruing to reinforced concrete structures, it became necessary to design such columns as would be of much smaller cross-section than those of the type heretofore prevailing. Two procedures were available: one was to use a substantial quantity of steel in the form of an ordinary steel column suitably designed for this purpose, and the other was to use the concrete in such a way as to justify a much larger working stress per square inch than that prescribed in the Building Code. These requirements are met in an eminently satisfactory way by the columns of the type used. All columns throughout the building are composed of an interior filling of concrete combined with steel angles latticed in the ordinary manner of built steel columns. The interior columns and most of those in the exterior walls are built with four angles, with the usual riveted lattice or lacing bars on the four sides of each column. In the case of a number of the exterior columns it was necessary, for the attainment of unavoidable structural ends, to use a column of elongated cross-section with eight steel angles of the same general type as those in the interior, which, with the concrete filling, form a column of two cells, so to speak, rather than one. The plans accompanying this paper (Plates LIII to LVI) show fully and clearly this arrangement of the combined concrete with steel angles and lacing bars. In all cases the steel angles, with the corners of the angles turned out, were spaced as far apart as the extreme outer dimensions of the completed column would permit.

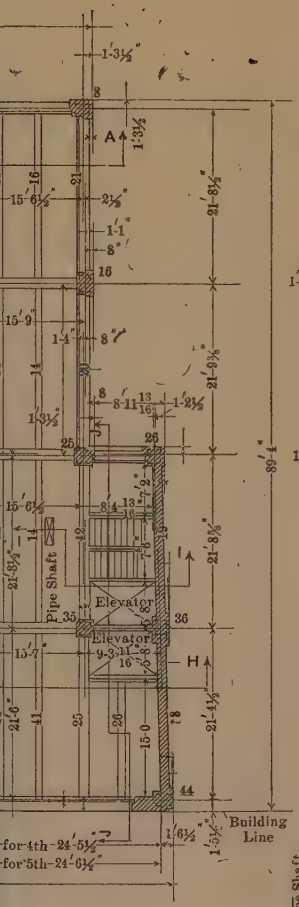
In accordance with the requirements of the Building Code, there is a thickness of 2 in. of concrete outside of the steel angles. This 2-in. shield of concrete is assumed to carry no load whatever; its function is simply that of fire-proofing, *i. e.*, to protect the steel against

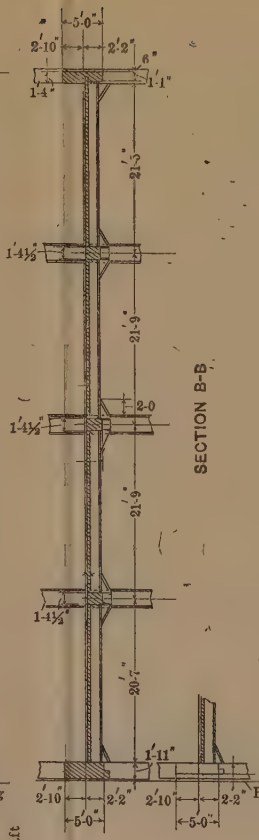
the immediate heat or flames of any fire that may start in the combustible materials at any time stored in the building, or of a conflagration in an adjoining building. The concrete within the exterior dimensions or outline of the steel angles is available for carrying a compressive or column load. As it is completely embraced or surrounded by the steel angles and lacing bars, it is steel "banded" in the most effective manner possible. Its enclosure in the steelwork of the column is so rigid, manifestly, that lateral strains under column loads must be greatly reduced—in fact, nearly prevented—within any ordinary limits of loading. Such concrete, therefore, is largely prevented from the usual yielding of that material under compression, and its compressive carrying capacity is increased. This is not only obvious from the condition of the concrete in these columns, but that view is confirmed by the comparatively few results of tests of concrete columns of this character. When, therefore, the plans of these columns were submitted to the Bureau of Buildings of the City of New York for examination and final approval, a special regulation was made permitting the concrete to carry a maximum working load of 750 lb. per sq. in. within the exterior limits of the steel angles, the exterior 2 in. of concrete, as stated previously, being considered simply a fire-protecting shield. This increased permissible load upon the concrete is coupled with the further provision that the cross-section of the steel in any column at any floor shall be sufficient to carry the entire dead load above that section without stressing the steel to more than 16 000 lb. per sq. in.

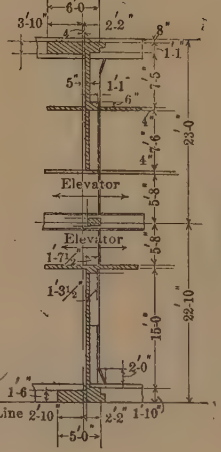
The use of the steel, in load-supporting condition, as a long column independent of the concrete, and at the same time forming a rigid banding member for the latter, with the consequent increase of permissible working load on the concrete, reduced the size of the columns in the basement and lower stories to dimensions quite consistent with the desired convenient and economical use of the clear floor space. Columns of this general type combine with their high carrying capacity great convenience in erection, for their steel sections may be erected ahead of the concrete work and afford convenient supporting members for the adjoining forms or for other erection work. The lacing bars, rivet heads, and other projecting column details act positively in creating a firm and complete hold or bond between the steelwork of each column and the concrete enclosed within it. This fea-

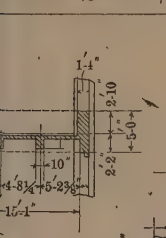

107'-0"
Pipe Shaft
5" hole
Elevator
Elevator
Building Line
for 4th Fl. 6"
for 5th Fl. 5"
for the 5th Fl.
for the 4th Fl.
126'-4"
SECTION A-A
SECTION C-C
SECTION D-D
SECTION E-E
Smoke Flue
Pipe Shaft
Elevator

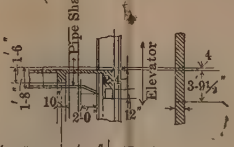
GENERAL PLAN OF
FLOOR ARRANGEMENTS
AND COLUMN LOCATIONS WITH
TYPICAL FLOOR SECTIONS.
Mc GRAW BUILDING
NEW YORK CITY

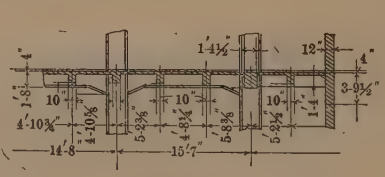


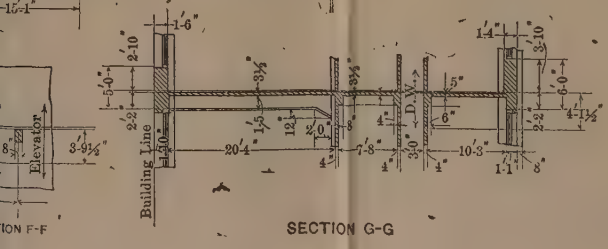












ture of these columns compels the steel and the enclosed concrete to act as a unit, and this action is enhanced by placing all the lacing bars in one direction inside the steel angles, the other set being placed outside, as shown on the plans.

The Building Code requires the ratio between the moduli of elasticity for the steel and concrete to be taken as 12. Hence, as the permissible compressive working stress in the concrete was taken at 750 lb. per sq. in., the corresponding working stress in the steel would be 9 000 lb. per sq. in.

The largest columns (in the basement) have exterior dimensions of 29 by 29 in., but, at the eleventh story, the exterior dimensions are reduced to 14 by 14 in. These columns were built in sections of a length equal to the combined height of two stories, *i. e.*, 25 ft. The extra metal involved in this procedure was too small to be of practical consequence, and the expense of half the joints, if a change of section had been made at every floor, was saved. Much time was also gained in the erection of the steelwork. The abutting ends of the column sections were faced, and the joints were made by suitable splice-plates. Where there was a change in the exterior dimensions of the steelwork, full-strength splices were made by riveting suitable short-angle sections on the interior of the splice-plates of the lower part of the joint. These details are also shown on the plans.

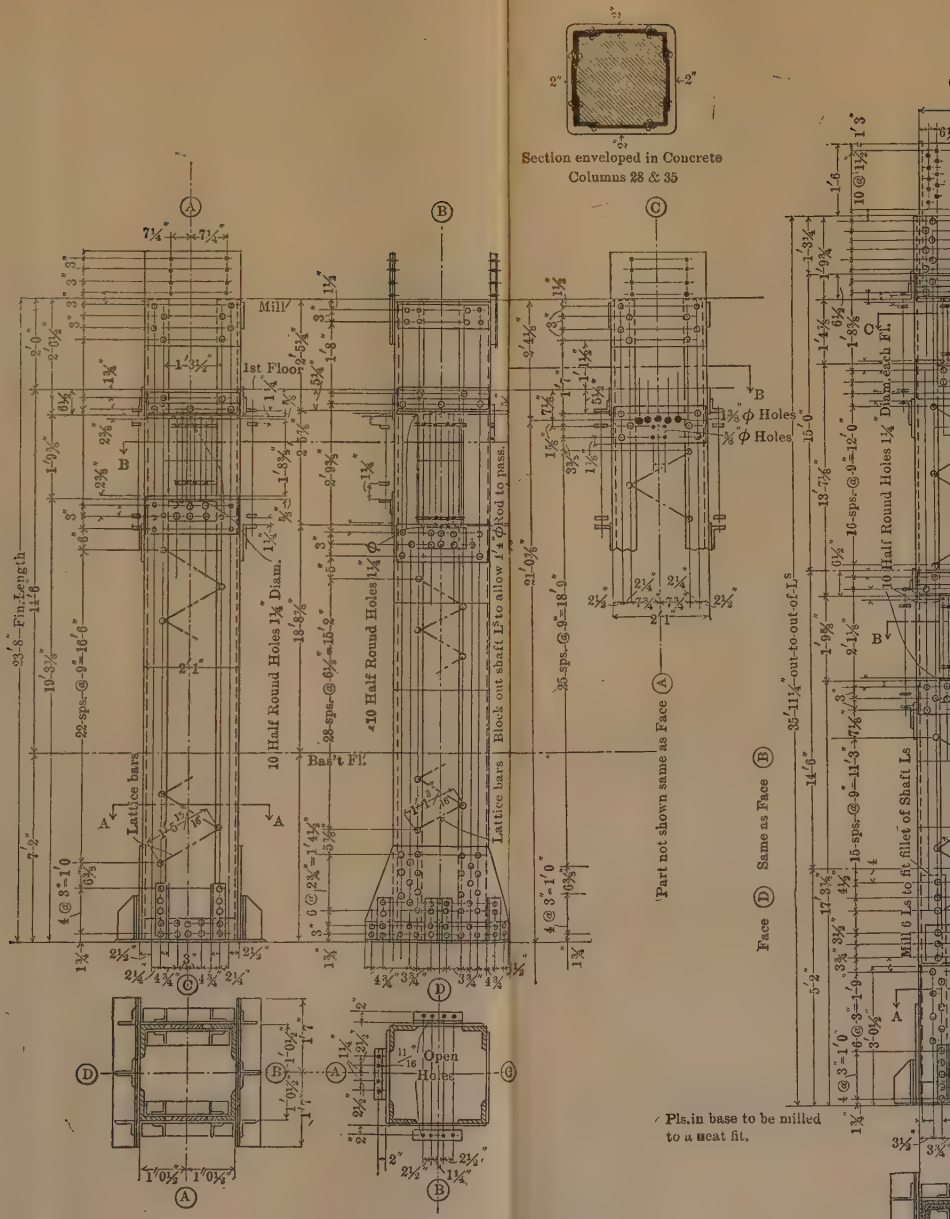
The ratio of the area of steel section to that of the concrete for the various columns varied from 10% in the basement, where the steel carries about 57% of the total load, to 3½% in the ninth floor, where 30% of the total load is carried by the steel. The requirements of the Building Code for a design of this type raises the percentage of steel to much higher values than in ordinary concrete-steel work.

Table 1 shows the number and sizes of steel angles used throughout the main columns of the building.

The design of the floors, in general, is quite similar to that usually found in buildings of this class, although there are certain important details which do not come under that observation. As the plans indicate, the spacing of the columns is such as generally to divide each floor into panels 21 ft. 9 in. by 14 ft. 8 in. between centers of columns, the clear span of the main girders between columns being 14 ft. 8 in., less the width or diameter of the column section. The clear span of the floor beams between the main girders is 21 ft. 9 in., less the width

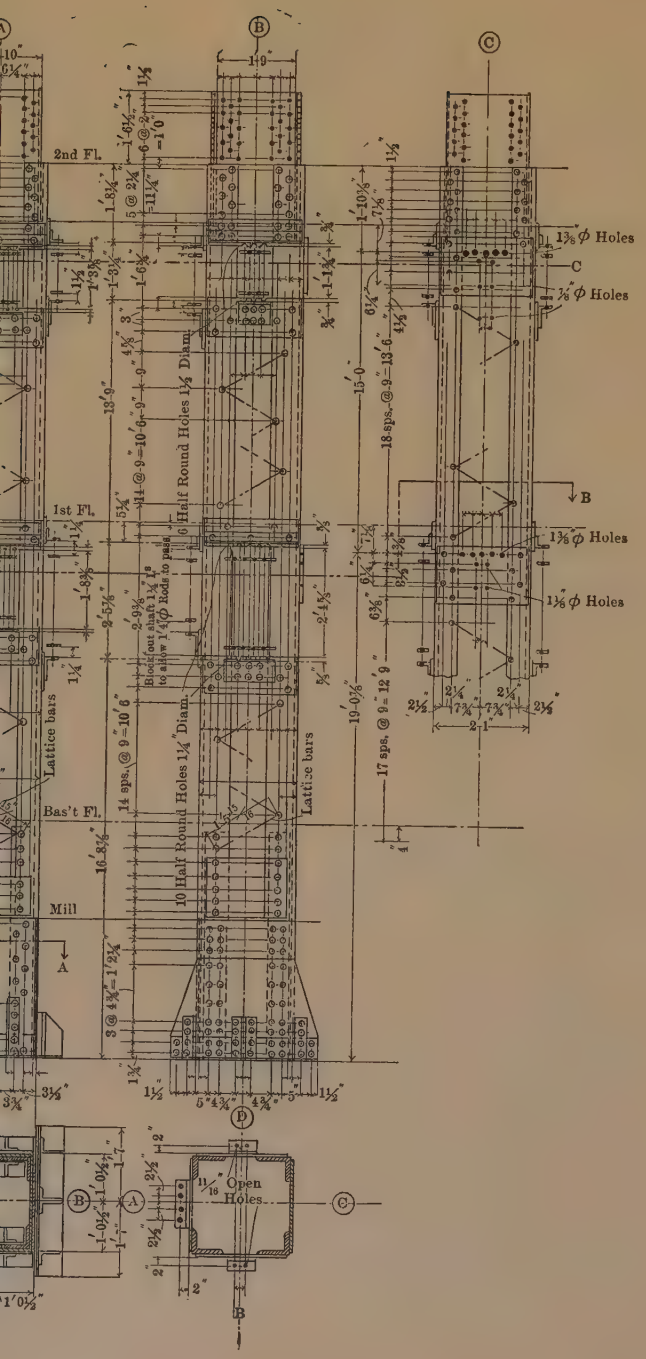
TABLE 1.—Number and Sizes of Steel Angles Used in the Main Columns of the McGraw Building.

Floor.	Sizes of Steel Angles in Columns, and outside dimensions of Steel Columns, *i. e.*, back to back of Angles.			
11....	4 L's, 2½ by 2½ by $\frac{5}{16}$ in. 10 by 10 in., b. to b.	4 L's, 3½ by 3½ by $\frac{5}{16}$ in. 10 by 10 in., b. to b.	4 L's, 2½ by 2½ by $\frac{5}{16}$ in. 12 by 26 in., b. to b.	4 L's, 2½ by 2½ by $\frac{3}{8}$ in. 12 by 26 in., b. to b.
10....	4 L's, 3 by 3 by $\frac{7}{16}$ in. 17 by 17 in., b. to b.	4 L's, 3½ by 3½ by $\frac{5}{16}$ in. 10 by 10 in., b. to b.	4 L's, 3 by 3 by $\frac{7}{16}$ in. 12 by 26 in., b. to b.	4 L's, 2½ by 2½ by $\frac{3}{8}$ in. 12 by 26 in., b. to b.
9....	4 L's, 3 by 3 by $\frac{7}{16}$ in. 17 by 17 in., b. to b	4 L's, 3½ by 3 by $\frac{5}{8}$ in. 17 by 17 in., b. to b.	4 L's, 3 by 3 by $\frac{7}{16}$ in. 12 by 26 in., b. to b.	4 L's, 3 by 3 by $\frac{9}{16}$ in. 12 by 27 in., b. to b.
8....	4 L's, 5 by 3 by $\frac{11}{16}$ in. 18½ by 18½ in., b. to b.	" " " " " "	4 L's, 3½ by 3 by $\frac{5}{8}$ in. 12 by 28 in., b. to b.	" " " " " "
7....	" " " " " "	4 L's, 6 by 4 by $\frac{3}{4}$ in. 18½ by 18½ in., b. to b.	" " " " " "	4 L's, 3½ by 3½ by $\frac{11}{16}$ in. 12 by 29 in., b. to b.
6....	4 L's, 6 by 4 by $\frac{13}{16}$ in. 20½ by 20½ in., b. to b.	" " " " " "	4 L's, 5 by 3½ by $\frac{5}{8}$ in. 12 by 30½ in., b. to b.	" " " " " "
5....	" " " " " "	4 L's, 6 by 6 by $\frac{13}{16}$ in. 20½ by 20½ in., b. to b.	" " " " " "	4 L's, 5 by 3½ by $\frac{11}{16}$ in. 16 by 32 in., b. to b.
4....	4 L's, 6 by 6 by $\frac{7}{8}$ in. 22 by 22 in., b. to b.	" " " " " "	4 L's, 6 by 4 by $\frac{11}{16}$ in. 16 by 33½ in., b. to b.	" " " " " "
3....	" " " " " "	4 L's, 8 by 6 by $\frac{7}{8}$ in. 22 by 22 in., b. to b.	" " " " " "	4 L's, 6 by 4 by $\frac{13}{16}$ in. 16 by 35 in., b to b.
2....	4 L's, 8 by 6 by 1 in. 24 by 24 in., b. to b.	" " " " " "	4 L's, 6 by 4 by $\frac{7}{8}$ in. 16 by 36½ in., b. to b.	" " " " " "
1....	" " " " " "	4 L's, 8 by 6 by 1 in. 25 by 25 in., b. to b.	" " " " " "	4 L's, 6 by 4 by $\frac{15}{16}$ in. 16 by 38 in., b. to b.
Basement....	4 L's, 8 by 6 by 1 in. 25 by 25 in., b. to b.	" " " " " "	4 L's, 6 by 4 by $\frac{15}{16}$ in. 16 by 38 in., b. to b.	" " " " " "


Section enveloped in Concrete
Columns 28 & 35
Mill
1st Floor
Bas't Flr.
10 Half Round Holes 1¼" Diam.
10 Half Round Holes 1¼" Diam. each Flr.
Block out shaft 1'-0" to allow 1¼" ϕ Rod to pass.
Lattice bars
Part not shown same as Face (A)
Face (D) Same as Face (B)
Mill 6 Ls to fit fillet of Shaft Ls
Open Holes
Pls. in base to be milled to a neat fit.

BOTTOM SECTIONS OF COLUMNS Mc GRAW BUILDING. NEW YORK CITY

PLATE LIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LX, No. 1075.
BURR ON
REINFORCED CONCRETE BUILDING.



of these girders. These prevailing lengths of spans of the beams and girders were modified at a few points in each floor to accommodate such features of construction or details of floor space as stairways, elevator shafts, and similar details.

As the plans indicate, all floor-girder and beam reinforcement was of round steel rods, of sizes running generally from $\frac{3}{4}$ in. to $1\frac{3}{8}$ in. in diameter. These rods were grouped in one plane on the tension side of each beam or girder. As a rule, every alternate rod was bent upward at the end of each span so as to rise within about 2 in. of the top of the concrete, from which point it continues through either the main girder or the adjoining column, as the case may be, into the adjoining span well toward the quarter point of the latter. By these means, true continuity of beams and girders was secured in every case. In addition to this, the end of each rod was bent down, forming a right-angled turn, with an arm from 2 to 3 in. long, thus insuring a rigid bond or connection with the concrete. This main detail, formed by carrying the rods through the girders and columns, is an important feature in securing continuity and rigidity in the general construction of the building. It is believed to be one of the most important details of the best design of reinforced concrete building construction, and it should be secured either in the manner adopted in this building or by some other procedure of at least equal excellence.

The proper spacing of these reinforcing rods was secured by suitable supporting details throughout the length of the beams and girders themselves and by notches cut in angle-brackets riveted on the columns where they joined the latter members. At the columns, rigidity of connection was secured by bolting clamps through the angle-brackets just named and jamming the rods by tightening the nuts against those brackets. This secured an exceedingly strong metal connection between the reinforcing rods and the steel columns, aside from the further rigidity produced by the concrete mass of the intersecting columns, beams, and girders. These details, shown on the plans, were designed with care for the purpose of securing the strongest possible steel connections between the beams or girders and the columns, it being one of the main purposes to attain rigid continuity between floors and columns, and floors and outside walls. It is believed that unusual stiffness and strength have been given to this building by these means.

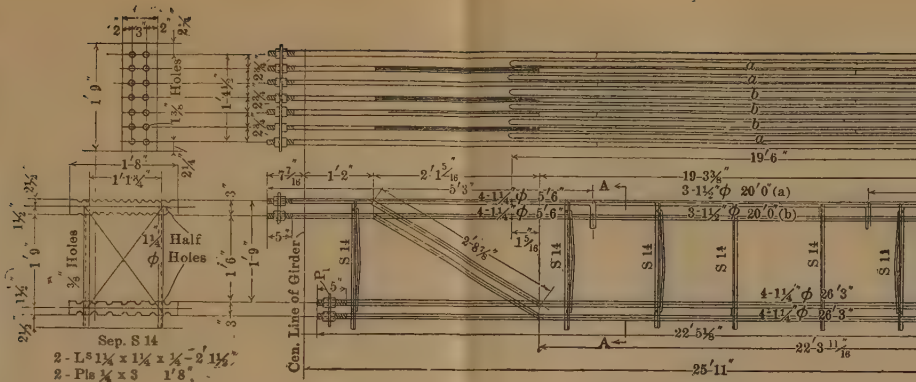
All beam and girder computations required by the floor designs

were made in accordance with the provisions of the Building Code of the City of New York, the usual common theory of flexure formulas for concrete-steel beams being used. While the parabolic law of variation of intensity of stress in concrete beams results in a trifling economy of material, it is less rational and simple than the usual straight-line law of variation, and the latter is more nearly accurate.

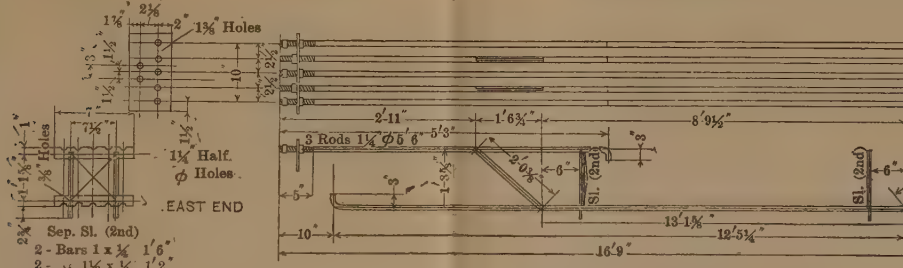
The Building Code of New York does not permit the condition of perfect continuity of beams to govern the design of reinforced concrete floor beams and girders. It is permitted, however, to consider the maximum bending moment of such beams, when uniformly loaded from end to end, as the total load multiplied by one-tenth of the span, rather than one-eighth of the span, as would be taken were the beams simply supported at each end. This is a widely-used method for continuity, in favor of which much can be said. It is extremely doubtful whether perfect continuity is attained in any case, but it is certain that a material advantage is secured over the condition of a beam simply supported at each end. The one-tenth rule, as it may be called, is a reasonable compromise.

Another condition insisted upon in the design of this building was a metallic provision for taking the end shears of beams and girders. By referring to the plans, there will be observed inclined portions of the round steel reinforcing rods to which attention has already been called. In every case there is sufficient steel in these inclined portions of rods to take the total end shears multiplied by the secant of the inclination rods to a vertical line without stressing the steel to an unsafe extent. While the tension in the steel produced in this manner, ignoring entirely the shearing resistance of the concrete, is higher than would normally be prescribed, it is far below the elastic limit, and forms a safe provision for the entire end shear in case any exigency should arise producing such a break in the concrete as practically to destroy its capacity for shearing resistance. In addition to this condition, there is sufficient concrete also at the ends of beam and girder spans to carry the shear at an intensity of 50 lb. per sq. in. of concrete section as permitted by the New York Building Code. This, also, has been considered one of the essential details of a concrete-steel building designed for a heavy and otherwise fatiguing service.

The floor slabs spanning the spaces between the floor beams are 4 in. thick in the lower floors, carrying the heaviest loads, and 3½ in.



GIRDER FRAME No. 33. (2d).



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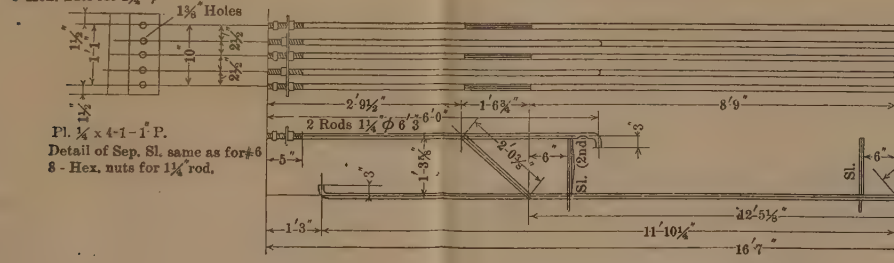

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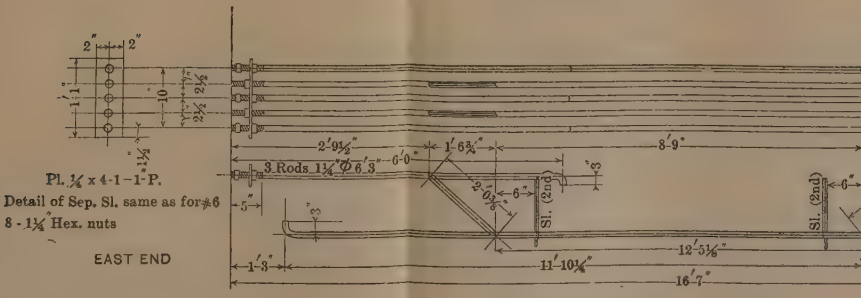
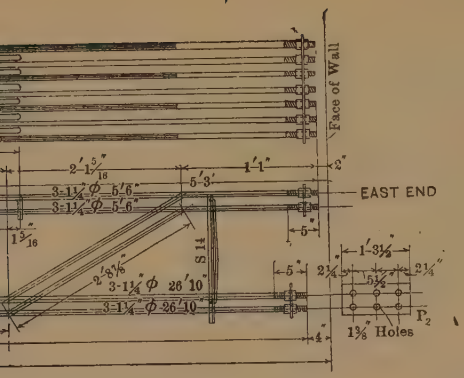
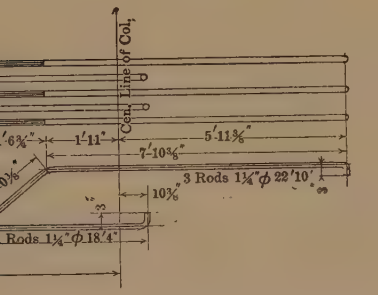
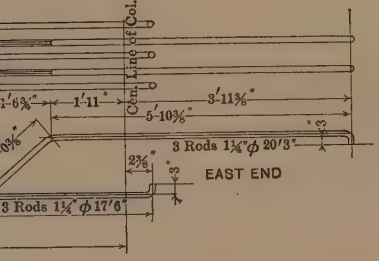


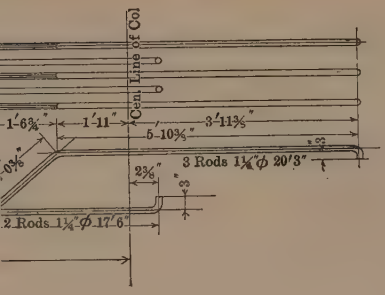
GIRDER FRAME No. 10. (2d).



REINFORCING RODS OF SECOND-FLOOR GIRDERS. Mc GRAW BUILDING. NEW YORK CITY







thick in all the higher floors. Their reinforcements are ½-in. and ⅝-in. rods, long enough to extend over a number of panels or spans so as to make these also continuous. Their general design is similar to that of the floor beams and girders. As the distance apart of the centers of the floor beams is about 5 ft. 2 in., the clear span of these floor slabs varies from about 4 ft. to a little more than 4 ft. 4 in., according to the thickness of the adjoining floor beams on either side of the span.

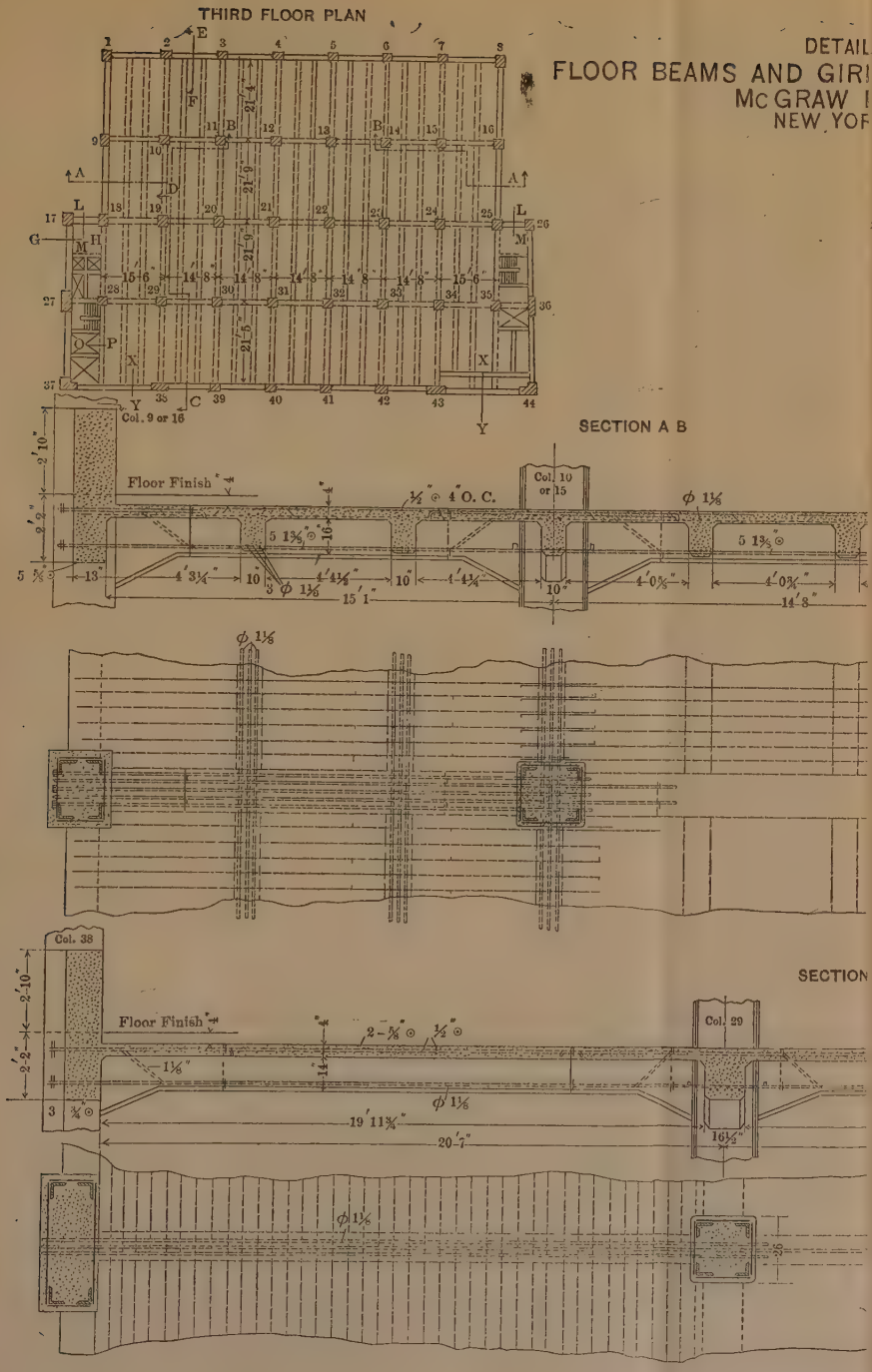
The proper design of the wooden forms or moulds for a concrete-steel building, in order to secure expeditious and economical work, is the most difficult part of the entire undertaking, and the principal improvements to be made in it are those which pertain to perfecting a proper system of construction of the forms and their ready handling. The quantity of lumber required in them, and the carpentry work necessary in making repairs consequent upon their use and re-use for successive floors, and in their supports, constitute far larger items of cost than might at first be supposed. If these costs are to be reduced, as they must be for heavy concrete-steel construction of the best class, the principal study of the engineer must be directed to this particular part of his work. While these ends may not be, and probably have not been, completely attained in this instance, the system of forms used gave excellent results in the quality of the concrete produced, and led to reasonable economy and efficiency. The weight of concrete and the relatively large quantities used in such individual members as beams, girders, and columns make heavy forms imperative and substantial support necessary.

The details of the timber forms for the floor beams and girders where they meet the column forms require especial attention. A proper design of the parts where the floors and columns join will result in great economy in the details of the forms. If the shapes of the exterior surfaces of the concrete are complicated and require careful fittings of the column and floor forms, expensive carpentry work will be required wherever a column pierces a floor, whereas simply shaped concrete surfaces will eliminate that work and greatly expedite the construction. Similarly, it is highly desirable that there shall be as few changes as possible in the exterior dimensions of the columns. If exterior column dimensions could be retained unchanged from basement to roof, it would make possible complete uniformity of the

details of column and floor forms throughout the entire building, eliminating a great amount of fitting and carpentry work otherwise unavoidable.

It is obvious that it is essentially impossible to retain uniform exterior column dimensions throughout the series of floors from the bottom to the top of the building, but the most scrupulous care should be exercised to make these changes as few as possible and in such a way as to reduce to the utmost extent changes of details in the forms.

A reference to the plans will show that the floor forms between the beams consisted of large shallow boxes with truncated corners between the sides and the bottom. They were placed bottom up on stable supports and separated by the thickness of the adjoining floor beams at the sides and by the thickness of the main floor girders at their ends. The bottom of the opening between the adjoining sides of any two of them was then suitably closed with planks or boards, so that, when the concrete was finally poured over their tops to the thickness of the floor slabs, the desired paneling of those slabs between the beams and girders was secured. It is imperative, for expediting the work, as well as for economy, that these box forms for the floors shall be constructed so that they may be removed readily after the concrete becomes sufficiently hard. To secure this important result, such forms must be readily collapsible at both ends and sides, and, at the same time, they must be substantial enough to hold the wet concrete without distortion, and so well made that they may be handled in removing from one floor and replacing on a higher one without sensible damage. This constitutes one of the most essential points in the design of these forms, which, in this case, were collapsible, although perhaps not as freely as might be desired. When the forms stick to the concrete in the process of removal laborers use sledges and iron bars, driving the latter between the new concrete and the forms and making a fulcrum of the former. This results in seriously marring what might otherwise have been a highly satisfactory concrete surface. The same general observations apply to the forms for the columns. Whenever the art of reinforced concrete construction is brought to the high state of excellence which it must ultimately reach through a proper design of the forms, making their erection, support and removal expeditious and free, the labor bills for the work and the repairs of the forms themselves will be so greatly reduced as to give this class of construction material economic advantage.


THIRD FLOOR PLAN
DETAIL
FLOOR BEAMS AND GIR
Mc GRAW
NEW YOR
SECTION A B
Col. 9 or 15
Floor Finish
Col. 10 or 15
Col. 38
Col. 29
SECTION

.S OF
DERS OF THIRD FLOOR.
BUILDING.
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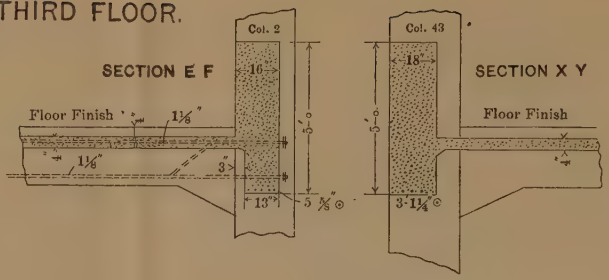

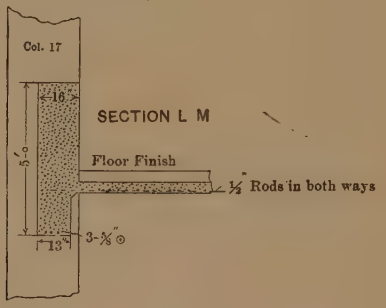

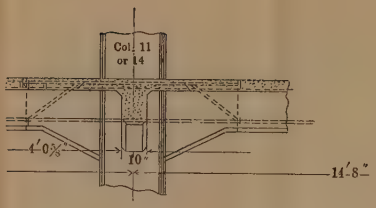

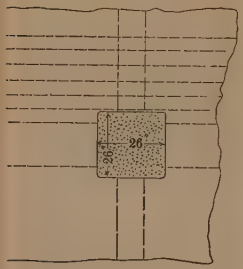

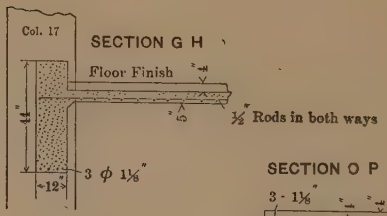

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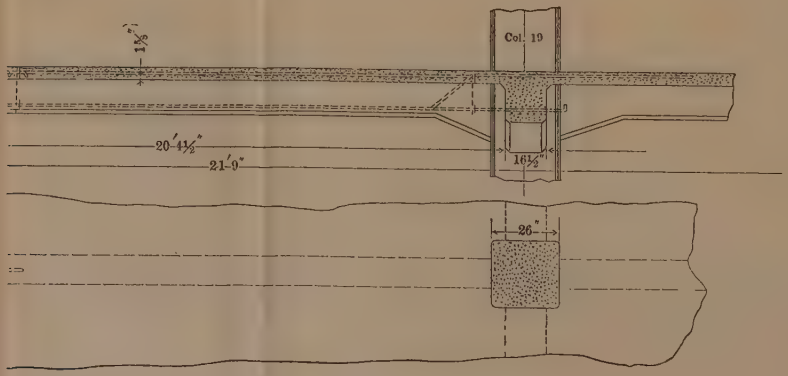


The length of time which the forms should remain in place supporting fresh concrete will depend on the temperature, and hence on the season as well as on the character of the work designed. It is clear that, with the substantial steel reinforcement of the columns of this building, a minimum of time would be sufficient for the column forms, but as it was not convenient to remove the latter until the floor forms were also ready to come down they were all kept in place for at least 19 days. Three entire sets of forms for floors and columns were made, so that, while concrete was being poured for the uppermost floor being constructed, the two stories immediately below were still supported by the timber forms. The lowermost set of forms was then taken down and placed above the freshly formed concrete last put in place. In this manner the forms could be left in place long enough to satisfy the requirements of even the winter season.

The concrete work of the building proper was begun in the basement in September, 1906, and the concrete parapet walls on the roof were completed on April 15th, 1907. These dates show that the work was carried on almost uninterruptedly throughout the winter. A few of the coldest and stormiest days of the winter were sufficiently severe to cause the work to be suspended for the day. When it is remembered that from the latter part of January until the early part of March the weather was phenomenally severe, it is demonstrated by actual experience that reinforced concrete building work may be conducted under proper conditions through a New York winter without material interruption.

After the completion of the lower two or three stories, when the organized force had become accustomed to the character of the work and the sequence of operations required, the average rate of progress, including the delays and occasional interruptions caused by the winter weather, was about 12 calendar days to a story.

From the early part of December to the latter part of March the window openings of the story below the floor in process of construction were closed with canvas, behind which, distributed over the entire floor, salamanders burning coke were constantly kept in operation. These salamanders were to some extent concentrated under the freshly poured concrete. By these means entirely satisfactory temperatures could be maintained, so that the retarding influence of the frost on the setting of the concrete was to a great extent eliminated, except for

the fact that occasionally the top surface of the fresh concrete was frozen. The warmest air produced by the salamanders would rise to and remain in the overhead cellular spaces of the timber floor forms and act there with a high degree of efficiency.

In connection with the operation of the salamanders below the fresh concrete, the top of the latter, as fast as poured, was always protected by a covering of hay or canvas, or both. These protective measures were scrupulously enforced throughout the winter, with entirely satisfactory results. Indeed, there was no evidence to be found throughout the whole building to show that any part of the concrete whatever was affected injuriously to the slightest extent by the frost. During a considerable portion of the winter a few salamanders were kept burning in the second floor below the work in progress.

All the construction work of the building was carried on from a high central temporary timber tower running from the basement to a height of nearly 75 ft. above the roof. This timber tower was 31 ft. square, and built with 10 by 10-in. yellow pine spliced corner posts properly braced. Each 10 by 10-in. corner stick carried a derrick boom 75 ft. in length. These derricks were first placed low down on the tower, and then raised from time to time to elevations required by the progress of the work. The booms were long enough to command the entire area of the work, and had a sufficient swing or reach to pick up material, including sections of the steel columns, delivered in the street in front of the building, and put it in its proper permanent place. The hoisting engines were placed in the basement, and steel cables ran from them up to the derricks.

It was a question at first whether the cost of this tower and derricks was justified by the amount and character of the work to be done, but they were found to be fully justified and well adapted to their purpose. It was probably as economical and expeditious a method as could have been devised for handling the materials and serving the work. As the building was carried up, the work within the limits of the tower was completed, with the exception of the points where the corner 10 by 10-in. sticks pierced the respective floors, where enough free room was left for the operating cables. After the concrete work was finished the tower was taken down through the succeeding floors of the building, and the holes left for the corner posts were filled.

There was nothing unusual about the character of the materials

used throughout the building. The material and workmanship of all the steelwork were supplied and manufactured under Cooper's Specifications. The Portland cement used was the Dragon Brand, and it was tested and supplied under the standard specifications recommended by the Special Committee of the American Society of Civil Engineers. The sand, broken stone, and gravel were supplied by different parties about the City of New York. Some of the broken stone came from the north, down the Hudson River, and most of the sand and gravel came from Long Island. Throughout the building proper, ¾-in. broken stone—trap rock and limestone—was used, with the exception of considerable quantities of gravel in which no pieces had a greater maximum diameter than about ¾ in. In some of the larger masses of the retaining walls and other similar parts of the foundation and basement of the building, 1½-in. broken stone was used.

The proportions of the concrete for the entire building were: one of cement, two of sand, and four of broken stone or gravel by volume. The consistency of the concrete was very nearly or quite wet enough to be that termed semi-liquid, so that it was truly "poured" into all forms for columns, walls and floors. Such a consistency of concrete is imperative for reinforced concrete construction of this class. It enables the concrete to form an intimate and dense matrix around the steel reinforcement, and produces a most excellent quality of material. While the concrete was being poured, laborers with long thin sticks continually agitated the fresh concrete in order to release all air bubbles and insure a dense and continuous product and the best possible bond with the embedded steel. There was no sensible excess of water in the concrete, but it was practically semi-liquid—too thin even to quake. The results throughout the entire work, in this respect, have proved to be in the highest degree satisfactory. The total quantities of the principal materials used were:

Cement	8 500 bbl.
Sand	3 000 cu. yd.
Broken stone	4 300 cu. yd.
Gravel	1 066 cu. yd.
Steel in latticed angle columns	655 tons.
Steel in round reinforcing rods	507 tons.

SPECIFICATIONS FOR CEMENT AND CONCRETE FOR THE McGRAW BUILDING.

Cement.—All the cement used in this structure shall be true Portland cement of the quality prescribed in the specifications of the Committee of the American Society of Civil Engineers. The tests required to establish the suitability of the cement will be in accordance with those prescribed in the American Society specifications.

Concrete.—The concrete shall be composed of cement, sand and broken stone, or gravel, mixed with clean water.

The sand may be natural sand or the finer product of the stone crusher. It shall be clean, hard, and preferably of varying sizes, so as to reduce the volume of voids. The gravel or broken stone shall be of varying sizes, free from sensible amounts of clay, loam, or foreign matter. The largest pieces shall not exceed 1½ in. in their largest dimension.

All concrete shall be thoroughly mixed so as to work the cement uniformly through the entire mass. All particles of sand, gravel, or broken stone must be completely coated with cement, and all the voids entirely filled.

All concrete shall be mixed in a machine mixer. The volume of matrix and the aggregate with the requisite quantity of water shall be kept in the mixer long enough to produce the intimate admixture desired. The quantity of water used shall be sufficient to make a wet or even semi-liquid concrete, so that it will readily run or flow into all the small spaces to be filled around all classes of steel reinforcement, whether in the floors or in the columns. This concrete shall be too wet to ram, but, while being poured and immediately thereafter, it must be actively agitated by long thin rods so as to expel all entrained air and produce a continuous and intimate bond with the steel embedded in it.

All concrete for the building proper, *i. e.*, above the foundations, shall be composed of such proportions of cement, sand, and gravel, or broken stone, as will make a perfectly solid mass, with a little surplus of cement over that required to fill the voids. The proportions of this mixture will be equivalent to 1 volume of cement, 2 volumes of sand, and 4 volumes of gravel, or broken stone.

In these provisions for the concrete it is the intention to take advantage of balanced sand, gravel, or broken stone, *i. e.*, to use varying sizes for the purpose of decreasing the voids, and using less cement

to secure the best results. It will probably be necessary, therefore, to experiment with different proportions of different sizes of sand and gravel, or broken stone, in order to ascertain the best mixture of the available materials at hand to secure the desired ends.

The largest pieces of the aggregate, *i. e.*, gravel, or broken stone, for the building proper shall not exceed $\frac{3}{4}$ in. in their greatest dimension. For the retaining or area walls around the basement of the building, and in foundation masses, the maximum size of broken stone or gravel in the aggregate may be $1\frac{1}{2}$ in.

The proportions of the concrete for the retaining or area walls and foundation masses shall be 1 volume of cement, 3 volumes of sand, and 5 volumes of gravel, or broken stone, the aggregate being balanced or graded so as to reduce to a minimum the voids to be filled with the cement.

DISCUSSION.

Mr. Tucker. H. F. Tucker, Assoc. M. Am. Soc. C. E. (by letter).—Professor Burr and the contractors who erected the McGraw Building are certainly to be congratulated upon having designed and built a structure of reinforced concrete of such unusual height. The fact that the building did not collapse during construction, and is even getting stronger with age, ought to be encouragement enough for prospective builders who, perhaps, have become unduly prejudiced against building material of this class.

It seems important to the writer, however, that the fact should be brought out clearly—for prospective builders and for the Engineering Profession as well—that the structural features of the building were not designed by an architect or by a contracting firm. In no way is this statement intended to claim for the Engineering Profession any credit which is not due, or to place it above either of the others. It is simply intended to point out the success attendant upon harmonious co-operation between these three distinct professions: the architectural, the engineering, and the contracting.

The writer believes that it is only by each keeping strictly to his domain that the perfect can be most closely approximated. It is true that the architect must know considerable about engineering, in order that he may design a possibility, but he cannot be both architect and engineer, nowadays, and excel in either. The engineer must know a great deal about construction, in order that he may design economically. A contracting firm should be purely a contracting firm, and not "contracting and engineering," though such a firm should employ competent engineers in its service, but not for the purpose of doing the work of the consulting engineer. Even in the cost-plus-a-fixed-sum system, where there should be no incentive to "skin," there is the temptation, if such a firm does the designing, to please the unsuspecting owner by providing a design based too evidently on cheapness rather than on proper strength.

It is the writer's firm conviction that, had the McGraw Building not been designed by an independent, unbiased engineer, the success of such an undertaking would not have been assured.

Professor Burr has brought out in his paper many points in reinforced concrete design which are frequently overlooked. The most important point is the study of the design with a view to saving money by duplication and simplicity of form work. In steel construction it is usually found more economical to run one section of column up two or three stories before making a change. So, also, in reinforced concrete construction, especially in a design of this type, where much of the stress is taken by the steel, it will be found cheaper to vary the

steel, and sometimes the mix, leaving the outside dimensions of the column unchanged for several stories. Mr. Tucker.

Uniformity, as much as possible, everywhere, even in depth and width of beams and girders, will usually result in a big saving in labor. On the other hand, irregularities produce what might be called a very large coefficient of friction in the carpenter's mind, and his speed is reduced to a remarkable degree. Simplicity and duplication in the reinforcing will result usually in a large saving. The fewer bends in the steel, and the more pieces that can conveniently be made into a unit, the better. It has always seemed to the writer that the bending of floor rods is not economical practice. It is hard to hold the rods thus bent from tipping over, and especially with round rods, which are cheap; two layers of rods, one in the bottom of the slab and one in the top, both continuous, seem to be cheaper, especially for thin slabs and short spans.

It appears from the details on Plate LVI that every other slab rod is turned up over the beams, thus giving only half as much steel to resist the negative bending over the beams as at the middle of the span, where, if the author had been calculating on continuity, the moment is less than $\frac{Wl}{10}$. If this is the correct interpretation of the drawings, it would seem that there is either a redundancy of steel at mid-span or an insufficiency over the beams.

The writer is in doubt as to whether it is economical to cast brackets under the ends of the girders and beams connecting to columns, as a brace against wind stresses, even in a high building, if it is as broad as the one under discussion. A deeper beam, if there is head room to spare, would simplify the forms, and make them more easily adapted to longer spans where the column section changes. When one sees a steel-framed building of ten or eleven stories and only 20 or 30 ft. from front to back, braced against wind by the ordinary web connection supplemented by 3½ by 2½-in. angle-lugs at the top and bottom of 20 or 24-in. I-beams, as was actually seen in Washington, D. C., he begins to wonder why a 30-in. reinforced concrete girder could not be connected to its column, far more stiffly and safely, by a few diagonal rods, and without the use of expensive brackets.

The removal of column forms, even with the substantial steel reinforcement in the columns of the McGraw Building, should not be reduced to the "minimum of time." To be sure, the lower columns would not have nearly all their dead load, but a column is a delicate thing, and really the backbone of such a structure.

In regard to such work being conducted successfully, under proper conditions, through a severe winter: The writer was connected in a professional way with a number of large reinforced concrete factory buildings for a manufacturing concern in Canada, last winter. Work

Mr. Tucker. was being pushed, as the contract time was urging the contractors to complete the buildings. It may not have been a severe Canadian winter, but it was far ahead of any Boston winter of the last two decades at least. The floors of the factory and shipping buildings were designed to carry a live load of 250 lb. In the spring the architect tested the shipping building floor "which suffered the most by frost, and it sustained a load of sand and stone of 450 lb. per sq. ft. without any deflection." It is probable that, through arch action in the load, or adjoining slab and beam action, the test did not show the actual strength of the floor; but, even so, it demonstrates the possibility of combating a cold climate successfully.

Mr. Douglas. W. J. Douglas, M. Am. Soc. C. E. (by letter).—In reading this paper the writer's attention was called, as it often is in considering reinforced concrete designs, to the apparent waste of steel when used as reinforcement for concrete-steel compression members. In this particular case, the writer refers to the columns, which are heavily reinforced.

According to the author, the lower columns contain 10% of reinforcement, which reinforcement carries 57% of the total load; while, at the ninth story, the reinforcement is reduced to 3½%, carrying at this point 30% of the total load.

On the basis of the assumptions stated, namely, 750 lb. compression for concrete and a ratio of moduli of 12, the steel in the columns can only be stressed to 9 000 lb., plus some small erection stress.

Now, considering the lower columns with their 10% reinforcement, assuming the cross-section of the concrete to be 625 sq. in., less about 56 sq. in. for the steel, it is found that the concrete in the columns is carrying a total load of approximately $569 \times 750 = 426\,750$ lb., and the steel, having a cross-section of 56 in., carries a total stress of 504 000 lb., making the total load on these columns 930 750 lb.

If, for this concrete-steel column, an ordinary steel column should be substituted, it would only require 58.2 sq. in. of steel. If it were possible, in the concrete-steel columns, to stress the steel up to 16 000 lb., only 31.5 sq. in. of reinforcement would be required, instead of 56.2 sq. in., therefore, there is a waste of 24.7 sq. in. of steel, or 83.9 lb. per lin. ft., which steel would cost about $3.36 per lin. ft.

In the ninth story, this amount, on the basis of the foregoing, would be reduced to about 60 cents. Assuming an average of $1.95 per ft. for steel wasted on account of its low compressive stress, there would be a total loss in the entire structure of about $13 600. (The writer, of course, acknowledges that this additional steel is an additional factor of safety, but the point that he is trying to bring out is, that it appears to be an uneconomical way of providing for a factor of safety.)

Mr. Douglas.

It is not the writer's intention to criticise the building in question, because he realizes that the circumstances were such that it was quite impossible to have saved this steel, yet he believes that in many buildings—and he is certain that in many bridges—it is possible to stress the steel in compression, both in columns and in girders, to about 16 000 lb. by designing the structure so as to make the steel in compression carry all or nearly all the compressive stress due to dead load, and he believes that this may be attained economically.

A case of a concrete-steel bridge (girder), now under construction in the District of Columbia, may be cited: On account of the limiting conditions, it was necessary to reinforce the compression chords of this bridge with a large percentage of steel. As the writer only stresses concrete in work of this class to 450 lb., he could not calculate that the steel in compression, under a design of the customary type, would carry more than 6 750 lb. per sq. in. The reinforcement consisted of two riveted trusses having a depth of about 6½ ft. The floor system also consisted of reinforced concrete, the beams of which were reinforced with built-up, steel I-beam sections, the full end shear in the beams being developed in the steel connections of the floor beams and main girders.

This type of reinforcement was selected by the writer for the following reasons: its ease of erection; the comparative certainty with which the stresses could be calculated; the certainty of having the steel located in the actual construction in accordance with the theoretical relation to the concrete; positive anchorages could be obtained for all members, particularly as to the connection of the web members and the chords; the girders have a small factor of safety in themselves; the forming work was simplified; the cost of depositing concrete was possibly decreased; and, because there was a great saving in the cross-section of the steel compression members on account of the practicability of stressing the steel in compression to approximately 16 000 lb.

The forms for the entire structure are to be suspended from the girder and from the floor beams. The design contemplates the jacketing of the floor system first, then the jacketing of the large girder, by which methods of procedure the steel girder will be made to carry almost the entire dead load, exclusive of the paving. The live-load stresses in the steel and the concrete will, of course, be distributed on the basis of the ratio of moduli.

In regard to the construction of large girders for bridges, which are exposed to the elements, it is thought that this method is a particularly good one because the amount of flexure in the concrete-steel girder is cut down to a minimum, therefore the possibility of the formation of incipient cracks on the tension flange of the concrete girder is also cut down to a minimum. Whether or not the formation

Mr. Douglas. of these tension cracks on exposed girders is a matter of import is, of course, a matter of conjecture rather than fact. It is believed that this principle may be applied to many buildings without unnecessary delay, and without an additional cost of forming.

In a second bridge, the writer is going to concrete the columns last, after the entire superstructure above the columns has been completed. In this way, the steel reinforcement, which consists of an ordinary steel column, will be made to carry the entire dead load.

Professor Burr, referring to the concrete in the columns, says: "As it is completely embraced or surrounded by the steel angles and lacing bars, it is steel 'banded' in the most effective manner possible."

The writer is rather inclined to believe that if the steel bands are not attached rigidly to the vertical reinforcement, a stronger column will be attained. His reason for this is that the concrete, in setting up, shrinks, particularly where the day's pour results in a long length of column, and soupy concrete is used, which is almost universal practice.

Now, as it is thought that this shrinkage takes place after initial set, it seems manifest to the writer that detached bands would give better results. As a matter of fact, it seems probable that the concrete adjacent to the vertical reinforcement will be injured more or less, on account of this vertical shrinkage, but this cannot be helped, and must occur even when plain round rods are used, unless they are mathematically straight. Therefore, it appears to the writer that, with the same cross-section of steel and concrete, the ordinary star column, with detachable steel bands, would be stronger.

Mr. Gayler. CARL GAYLER, M. AM. SOC. C. E. (by letter).—The McGraw Building differs from the numerous purely reinforced concrete buildings which have been constructed all over the country principally in the type of the columns used. As described in the paper, these columns are built of steel angles, laced on all four sides and filled with concrete, and have been proportioned on the assumption, not only that both materials carry the loads jointly, but, furthermore, that through the lateral support afforded by the steelwork, the strength of the concrete is increased to such an extent that a unit pressure of 750 lb. per sq. in. —more than twice the pressure per square inch allowed by the Building Department of the City of New York on reinforced concrete columns—was deemed permissible. The unit pressure on the steel was then determined according to the ratio of the moduli of elasticity of the two materials (1:12). Such unit stresses in the columns permitted a height of eleven stories for the McGraw Building, with reasonable sizes for the columns, even in the lower stories and basement.

Now, it is clear that if these assumptions were correct, that is, if the two materials of which the columns are composed were to act as a

unit, with the additional advantage of greatly increased carrying capacity of the concrete, the superiority of such columns over the reinforced concrete column, in regard to reduced areas of cross-section, and safety, and over the steel column in regard to economy, would be so great that buildings constructed on the same plan, and of great height, would soon be generally built and become strong competitors of the steel sky-scraper. But, are the claims for these combination columns justified? Not once in this paper is there mentioned the fact that concrete, in drying in air, shrinks. Waiving the question of relative changes in the dimensions of the concrete and of the steel under changes of temperature, or of the relative compressibility of the two materials under loading, the thoroughly established fact of the contraction of concrete in the process of hardening and drying remains. The phenomenal increase in the use of concrete for engineering works during the last ten years could not have taken place unless full account had been taken of this important quality of concrete. The exact amount of this shrinkage is unknown, but that the changes in the volume of concrete are considerable, probably greater than those under subsequent changes of temperature, is just as sure as that no corresponding reduction in the size and length of the steel column, put up in the building ready to receive the wet concrete, will take place. No amount of care taken in pouring and stirring will alter this inherent characteristic of all concrete used above ground. Mr Gayler.

Thus, in the McGraw Building, there are composite columns, about 150 ft. long, of varying cross-section, consisting of an outer shell of steel and filled with a material which, as any engineer who has had any experience with concrete knows, will at certain unknown distances develop shrinkage cracks. That the surface contact between the steel and the embedded concrete will have an influence on the manner of shrinkage, and therefore on the frequency and position of these cracks, is true, but to suppose that the latter are obviated through this contact is equivalent to assuming that the steel plates, angles, and rivet heads will, through their mere contact, stretch the enclosed concrete, uniformly, throughout the aggregate height of the eleven stories of the building. This would be a bold position to maintain, the more so because the contact between the steel and the concrete is not at all uniform: Throughout the greater part of the length of the columns the concrete is confined by angles and lace bars, with the rivet heads spaced some distance apart, while at the joints of the steel columns and, to some extent, at the points of lateral connections, it is solidly enclosed by angles and plates with closely spaced rivet heads.

To consider the influence of the inner surface of the rigid steel columns of the McGraw Building on the enclosed concrete, as far as shrinkage of the latter material is concerned, equal to, or even greater than, the effect of shrinkage rods of the ordinary reinforced concrete

Mr. Gayler. work, is a fallacy, because the shrinkage rod, owing to its light cross-section, partakes to some extent of the deformations of the concrete, and, owing to its position inside of the mass of the concrete, is able to take up the stresses induced by the changes in form of the concrete, neither of which conditions is fulfilled by the steel shell. By deformations of the concrete is here meant not only the reduction of length, but also reduction of cross-section in the process of drying. The writer considers it anything but an extravagant statement to say that, in places, the concrete after hardening will not cling to all the four inner sides of the steel column.

To assume that the concrete contracts in such a manner that it forms a monolithic column, as high as the building, and bearing on the foundations in the basement; in other words, that the mode of vertical contraction of the concrete takes place as if there were no contact with the steel at all is, of course, out of the question, and could not be maintained by the designers of this building for one moment, as in the paper great stress is laid on the "firm and complete hold or bond between the steelwork of each column and the concrete enclosed within it." The lace bars have been specially arranged so as to counteract such independent movement of the concrete as much as possible.

The above stated considerations have led the writer to the following conclusions:

1.—The McGraw Building is a reinforced concrete structure with composite columns, the carrying capacity of which columns consists in the steel.

2.—The concrete filling of the columns is an excellent provision for the prevention of rust, highly advantageous, together with the concrete casing of the columns, in case of fire, besides affording some additional rigidity to the latticed portion of the columns.

3.—The wise provision, insisted on by the Bureau of Buildings of the City of New York, "that the cross-section of the steel in any column at any floor shall be sufficient to carry the entire dead load above that section without stressing the steel to more than 16 000 lb. per sq. in.," is, in view of the great weight of the building, a guaranty for the reasonable safety of the McGraw Building.

Mr. Jamieson. J. A. Jamieson, M. Am. Soc. C. E. (by letter).—Referring to the columns built of four steel angles, latticed diagonally, and the interior filled with concrete: The writer cannot agree with the author's claim that "the concrete is 'banded' in the most effective manner possible." In fact, he believes it to be obvious that the steel members of this column, as built, cannot possibly retard the lateral swelling of the concrete due to the compressive load upon it.

Assuming 12 to be approximately correct for the ratio between the moduli of elasticity for steel and concrete, and also assuming that each of the materials will take its computed share of the load, there will be an equal shortening of both the concrete and steel angles.

Mr. Jamieson.

Now, it is well known that the transverse expansion of concrete under compression is very much less than the longitudinal shortening. Under a working stress of 750 lb. per sq. in., the former is probably not greater than 20% of the latter. With the lattice bars set at an angle of 30° from the horizontal, as the steel angles shorten under compression, these lattice bars, with their rivets acting as pivots, will force the angles apart, and the transverse expansion of the steel members will be much greater than the concrete, under which condition it must be obvious that the steel cannot band or restrain the concrete and thereby increase its power of resistance. In addition to this, it would be expected that the bond or adhesion between the steel and concrete would be destroyed.

If the steel angles had been connected by horizontal bars or battens, a much more efficient column would have been obtained; it would still, however, be far from "banded" in the most effective manner possible, since the thin straight bars, spaced at considerable distances apart, would not present material resistance to the lateral swelling of the concrete.

The column shown by the author cannot properly be called a reinforced-concrete column, but should be termed a composite concrete and steel column, having a strength in direct compression equal to the sum of the resistance offered by the steel and the concrete, using 12 as the ratio between the moduli of elasticity of the concrete and the steel, as required by the New York Building Code. Since, however, the concrete was placed in the work in a semi-liquid state, there would be considerable shrinkage during the setting and hardening period, which would most probably produce corresponding initial compressive stress in the steel and tensile stress in the concrete, and this would materially increase the load on the steel and reduce the load on the concrete, and most probably cause the load ratio to become nearer 16 than 12.

The writer fully believes that reinforced concrete, as a structural material, is particularly well adapted for such buildings as the one under discussion, and does not contend that the column, as built, and under working stresses of 750 lb. for 1 : 2 : 4 unreinforced concrete, and 9 000 lb. per sq. in. for steel, is an unsafe one; in fact, he believes it to have an ample factor of safety, but he contends that the assumptions on which it is based are theoretically untenable, and are entirely at variance with all reliable information thus far obtained from tests.

The compressive member, or column, is probably the most difficult problem with which one has to deal, in the majority of structures, and it may not be out of place to venture the opinion that, up to the present time, the only rational reinforced-concrete column produced, in which the stresses are capable of being analyzed and closely calculated, and the one which gives the highest efficiency for the materials

Mr. Jamieson. used, is that type in which the concrete is used in direct compression and the steel in tension only, designed to resist the lateral expansion of the concrete, thereby greatly increasing its power of resistance under compressive loads.

The tests conducted by M. Considère very fully proved the efficiency of this type of column, when properly designed and built. Subsequent tests conducted by A. N. Talbot, M. Am. Soc. C. E., have broadly confirmed the results obtained by M. Considère. Professor Talbot's tests, however, appear to indicate that the hooping remains inactive until a considerable load has been applied to the column, but it is believed that this is entirely due to the lateral shrinkage of the concrete, owing to insufficient water being attained, during the hardening period, to insure full crystallization.

It is true that many irrational designs of columns of this type have been produced and used; notably, those having a considerable percentage of longitudinal rods, with hoops or bands spaced at too great distances apart to prevent the lateral swelling of the concrete, with the consequent deflection of the rods and very high stresses in the bands, or those having spirals wound at too great a pitch, which increase in diameter with the shortening of the column, and also leave the concrete free to swell between the hoops. The hooped column, however, is gradually becoming better understood, and is being studied by many able engineers, and one may look forward to material improvement in the future.

Mr. Smith. Walter M. Smith, M. Am. Soc. C. E.—About three years ago, the speaker had occasion to observe the condition of stress of reinforcing steel bars in concrete. In constructing a battery of high-power guns at Charleston, S. C., the ceilings of the rooms had "Johnson" corrugated bars placed in the concrete, about 4 in. above the surface. The thickness of concrete from the ceiling to the top was about 10 ft., but at a height of about 2 ft. above the ceiling a water-proofing layer of asphalt was placed separating the concrete above from that below. The span, in the direction of the reinforcing bars, was about 11 ft. Bars were placed of a sufficient size, and at a proper distance to take all the tension, with the 2-ft. thickness of concrete acting as a set of beams 11 ft. in length. The bars were computed to be stressed in tension at 16 000 lb. per sq. in.

A few months after this concrete had been finished, it became necessary to cut a recess in the ceiling to a depth of about 1 or 2 in. beyond the reinforcing bars and about 2 ft. long in the direction of their length. When the concrete was cut from around the bars they sprang out of line, showing that they were in compression instead of in tension. In the speaker's opinion, the only way to explain this is that the concrete in setting had shrunk sufficiently to put a considerable compressive stress in the steel. The concrete in the lower part of the

beam, therefore, was taking all the tension, and the steel could take none until the tension in the concrete became sufficient to cause it to stretch to the position it occupied when constructed. When this point is reached the steel begins to take the tension, and at a constantly increasing ratio as the concrete begins to develop fine hair-like cracks extending gradually upward from the bottom. The depth of concrete was very great in proportion to the span, in this case, therefore the proportion of steel was very small. The concrete on the tension side of the beam was not stressed very high in carrying the load, without considering the steel. Mr. Smith.

This is the only instance in the speaker's knowledge where concrete has been cut from around reinforcing bars and thereby a chance given to observe the condition of stress in the bars. He is glad, therefore, of this opportunity to bring it to the attention of engineers.

In view of the foregoing facts, it seems to the speaker that it is somewhat risky to construct long columns, as in the McGraw Building, with a large percentage of steel, and expect the concrete to take any portion of the load to the foundation. In the speaker's opinion, the concrete, being prevented from contracting longitudinally by the steel, will be in tension although it may show no appreciable cracks, and cannot, therefore, be relied on to take its due proportion of the load to the foundation. In the hooped column, if only enough longitudinal reinforcement be placed to hold the hoops in position during construction, the concrete is free to contract in setting and, therefore, is not in the same condition of stress.

The speaker believes that the fact mentioned by Mr. Goodrich, of the shrinkage of the concrete in the center of columns, reinforced as these are, goes to prove that the steel reinforcement does prevent the shrinkage of the concrete during setting and leaves it in a condition of tension.

Clarence W. Noble, Assoc. M. Am. Soc. C. E. (by letter).—The writer has always been of the opinion that insufficient consideration has been given to the question of the proper division of the total bending moment into the positive moment, provided for in the center of the span, and the negative moment, provided for over the points of support. Every question, connected with the moment of resistance of a concrete beam, has been discussed until there is practicable agreement as to the actual conditions prevailing, and the only discrepancies of any moment are those arising from the fact that certain engineers, in their desire for simplicity, knowingly, prefer to ignore various phases of the matter. A far larger percentage of variation in the practice of different engineers is accounted for by the variation in the division between the positive and negative bending moments than in any other single consideration. In fact, this matter has come to the point Mr. Noble.

Mr. Noble. where practice is now being crystallized by the adoption of building laws in various cities almost without any general engineering discussion of the question.

Obviously, with a uniform load distributed over several adjoining spans, there are a number of ways in which the total bending moment may be met. The system as a whole may be regarded as a series of simple beams, reinforced entirely on the bottom. It may also be regarded as a series of balanced cantilevers, extending out on each side from each point of support, and meeting at the center of the span, the reinforcement in this case being entirely at the top of the beam. By varying the quantity of steel at the bottom and top of the beam, it may be divided so as to be a combination of these two systems, with points of contraflexure located arbitrarily at the option of the engineer.

That this last statement is true, may be seen readily by a moment's consideration. The point of contraflexure in the first case—that of a series of simple beams—is directly over the point of support for each of the two adjoining spans. There is, in this case, no curve convex upward, and a cusp occurs at the point of support. This, of course, means that when fully loaded, a crack may be expected there. In the second case—that of a series of cantilevers—exactly the reverse takes place. There is no curve concave upward. The points of the contraflexure are concentrated at the center of the span, forming a cusp; and, under fully-loaded conditions, a crack will occur at the bottom of the beam at this point.

Now, by removing a portion of the negative reinforcement over the points of supports, these two superimposed points of contraflexure will be drawn apart and will approach the columns, and positive reinforcement will be required in the center of the span, in order to give the beam action over the interval between these points. Just how much positive reinforcement is needed will depend on the remaining amount of negative reinforcement.

As is well known, wherever the system of continuous beams extends indefinitely in either direction, the negative bending moment is represented by the formula, $\frac{l}{12}$, and the positive bending moment by $\frac{W\,l}{24}$. This condition, however, only takes place when both the negative and positive bending moments are adequately met by moments of resistance, and then only when the unit stresses in the material furnishing this amount of resistance are the same at the center span and the points of support. As conditions vary from this, say by decreasing the moment of resistance over the point of support, thus bringing the point of contraflexure toward the ends of the clear spans, the curvature over the point of support is increased, and consequently the unit stresses at this point are increased.

Mr. Noble.

Only one condition is theoretically imposed for the division of bending moment between positive and negative points. This, as the total moment, must equal $\frac{Wl}{8}$. The complement of $\frac{Wl}{10}$, of the center span, as demanded by the Building Laws of New York City, is $\frac{Wl}{40}$ at the point of support. This condition is actually required in French practice, although it seems to be largely ignored in the United States. If $\frac{Wl}{12}$ is used at the center of the span, theoretically, then $\frac{Wl}{24}$ must be used at the point of support; and if $\frac{Wl}{16}$ is provided for at the center of the span, theoretically, $\frac{Wl}{16}$ must also be provided for over the point of support.

Actually, one cannot "sail so close to the wind." If the French practice is followed and $\frac{Wl}{10}$ is provided at the center of the span and $\frac{Wl}{40}$ over the points of support, and if at any time actually theoretical conditions are realized, the reinforcement over the point of support will be stretched far beyond its elastic limit. Consequently, the actual bending moment at the center of the span will become $\frac{Wl}{8}$. It is possible that this statement should be "taken with a grain of salt," owing to the fact that the beam may be incorporated into the general floor system in such a way that, should an actual condition represented by $\frac{Wl}{8}$ occur, the extreme bottom fibers of the beam would be confined so much at the ends that they would not have opportunity to elongate, and therefore an internal arching effect would be set up. If $\frac{Wl}{12}$ be adopted at the center of the beam and $\frac{Wl}{24}$ at the point of support, obviously the unit stresses over the point of support will be much greater than at the center of the span, thus bringing them probably very close to the elastic limit. If the actual theoretical condition is adopted, that is, $\frac{Wl}{24}$ at the center of the span, and $\frac{Wl}{12}$ at the point of support, the use of T-beams is prevented, as the section will not reverse, and it would be necessary to increase largely the depth of the beam at the columns and for some distance on each side. For this practical reason, the writer knows of no work in which this set of coefficients has been used.

Mr. Noble. There is another consideration, also, which prevents too close an adherence to any theoretical division of the total bending moment of $\frac{W l}{8}$. This is due to the fact that there are conditions of partial loading in which greater bending moments can be brought about in various points of the system than under strictly uniform load. If, for example, the beam extends over several panels, and the load is only imposed on alternate panels, intervening ones being occupied by aisles, it is questionable whether there would be any positive moment in the aisles, and it is certain that the bending moment in the centers of the spans taking the load would be considerably greater than $\frac{W l}{24}$. If provision is made for $\frac{W l}{12}$ at this point, therefore, and for $\frac{W l}{12}$ also, at the point of support, a system is adopted which will never meet the theoretical conditions occurring at the same time in actual practice, but which will meet any possible stresses occurring at any point in the system due to the variation of this theoretical condition. The resulting beam also has the advantage of being uniform in section.

Mr. Waite. Guy B. Waite, M. Am. Soc. C. E. (by letter).—Had the McGraw Building been designed ten years ago, it would not have been built of reinforced concrete. Few engineers at that time had sufficient confidence to undertake the experiment, and the local building laws did not permit it. Even more recently than this, nearly every architect of importance in the vicinity would have refused to listen to an argument for using concrete in a building in which there was to be heavy vibration, as it was only considered fit for light, cheap buildings.

At that time, even engineers were afraid to speak in favor of concrete for general building purposes, for fear of becoming unpopular. One who dared engage in concrete building construction had to endure the humiliation of seeing some of his former friends pass quietly to the opposite side of the street when they saw him, in order to avoid meeting one who was engaged in an immoral business, and who was just escaping the meshes of the law. The change in public opinion, with regard to reinforced concrete, has been brought about purely by the merits of the construction.

About ten years ago the Building Department of New York City inaugurated standard tests for concrete constructions to be used for fire-proof floors. All constructions for floors had to be submitted to a 4-hour fire test, at an average temperature of 1 700° fahr. They were to be loaded with 150 lb. per sq. ft. on a full-sized floor not less than 14 ft. long. Immediately following the fire—the material being still red-hot—a regulation stream of cold water was to be thrown on the construction, with a pressure of 60 lb. per sq. in., for 10 min.

FIG. 1.—CINDER CONCRETE TEST HOUSE AFTER FOURTH FIRE.

FIG. 2.—STONE CONCRETE HOUSE, WITH TEST LOAD,
AFTER FOUR-HOUR FIRE TEST.

The construction was then to withstand a distributed load of 600 lb. per sq. ft. Mr. Waite.

During the five years following the inauguration of this test, about twenty-five concrete constructions withstood it successfully. By this time some of the public had been convinced that concrete possessed merits, as a fire-proof material, but did not dare to speak out; while others feared that it had some merits, and set out to kill it. Quite successful obstacles were placed in its way, by the Board of Insurance Underwriters, who fined it; by codes of law, which practically ruled it out; by labor unions, which dictated by whom and how it should be made; and by politicians, whose interests were generally in other directions.

Interest was finally awakened, by the favorable showing made by concrete in some of the great fires, on which a few honest reports were made by eminent engineers, and since that time concrete constructions have gained rapidly in popularity.

It was only about four years ago that the writer secured, from the Department of Buildings of New York City, the first permit ever granted in the Borough of Manhattan for a concrete building, including concrete wall construction. All who are interested in the advancement of reinforced concrete must feel indebted to Professor Burr for giving his name and influence to this cause.

The McGraw Building was undoubtedly made in reinforced concrete because it offered the best construction to withstand the peculiarly heavy work of a printing house; and because it gave the safest fire risk. The National Board of Fire Underwriters, some two years ago, recommended a minimum rate of insurance on similar constructions, and, from recent inquiry of the Local Board of Fire Underwriters, it is learned that even this august body has at last reached a point where a similar action is under consideration.

As a fire risk, concrete structures offer the best possible investment, either for individuals to carry their own risks, or for insurance associations to make a specialty of these constructions. Such undertakings would be the safest and most profitable kind of insurance ventures.

Concrete is superior to burned clay, not because it is more fireproof, but on account of its superior elasticity under the stress due to fire and water. Some years ago the writer constructed, entirely of cinder concrete, a test house, 14 by 14 ft. and about 12 ft. high, with walls 12 in. thick, and with a ceiling only 1½ in. thick. In various tests conducted by the Department of Buildings in that house the ceiling was submitted to four separate series of fire tests.

When this structure was torn down, to make way for dock improvements, this 1½-in. ceiling was examined by an engineer from the Department of Buildings of New York City, and by Professor Wool-

Mr. Waite. son, of Columbia University, and was found to be in good condition, the fire having affected scarcely ½ in. on its under side, and this was due to the first fire.

While reinforced concrete has been demonstrated to be superior in many respects to other forms of fire-proof construction, it has forced its way to the front principally on account of the economy it has effected.

The parts of a building in which it is best adapted must be determined largely by the engineer's experience. Sometimes this experience is paid for very dearly, and forms a secret chapter in his biography.

Local conditions often alter circumstances to such an extent that a kind of construction which might be erected in one locality at a profit would become a loss in another locality a short distance away.

As the relative prices of built steelwork and concrete per unit section are about as 65 to 1, and the relative working capacities in compression (16 000 to 500) are about as 30 to 1, it is evident that, other things being the same, the more concrete is substituted for steel in compression, the greater is the economy.

Where heavy loads are to be carried, concrete will be found highly advantageous; conversely, where small loads are carried, it will have little advantage. Keeping this fact in view, one would expect to find the greatest economy in using concrete for column supports and floor constructions—the heavier the construction, the greater the economy.

The great barrier to using concrete for columns is the impractical size necessary when more than a few stories are required. Leaving to others the discussion of the rationality of the combination of concrete and steel in the columns of the McGraw Building, the writer considers this form of construction superior to anything heretofore done in reinforced columns.

The column is reduced to a reasonable size, and is made safe against accidents. The positive dead loads from the building are carried by positive steel supports, while the doubtful superimposed floor loads are adequately provided for by the more questionable form of concrete reinforcement.

The certainties are balanced one against the other, and the uncertainties are also brought to face each other. Although columns of this form are not directly the cheapest, the writer believes them to be the most economical, all things considered. Columns of this form are adapted to much speedier erection than the cheaper reinforced concrete, and are absolutely safe during erection; they also allow as good a monolithic connection of the column with the floor as that obtained in other reinforced forms. The column forms, with the steel as a guide, cost less than where they are made as independent structures.

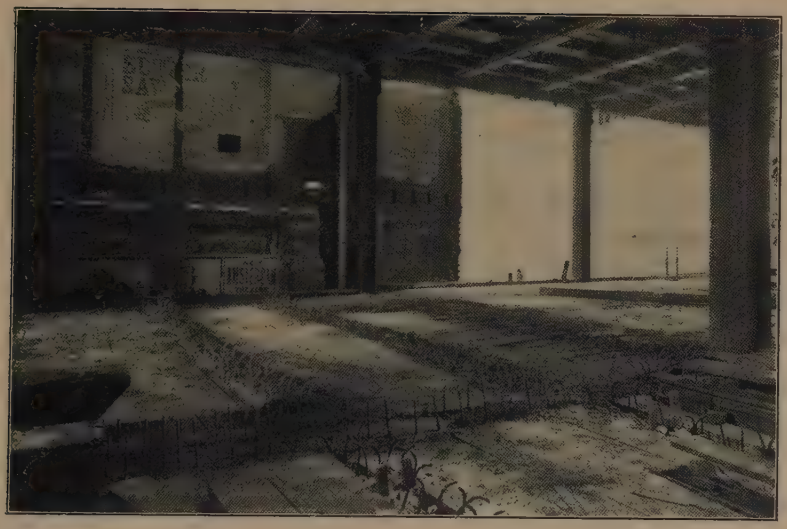


FIG. 1.—THE BONWIT-TELLER BUILDING.

FIG. 2.—THE SALVATION ARMY WAREHOUSE.

Mr. Waite.

As pointed out by the author, the construction of the forms is probably the greatest problem in the practical construction of reinforced concrete. Almost every beginner in this field has arrogant confidence in his ability to eclipse everything previously done in the way of perfect centering. It is only necessary to watch such an one and see a second scheme in his second job, a third scheme in his third job, and so on, until he becomes a meek plodder along the tow-path of experience.

A discrimination should be made between centers or forms designed for a building, and a building designed for the forms. Forms made to fit a special building may cost several times as much as those with which a standard building might be made. The cost of centerings may be reduced about in proportion to the standardization of the building. Many useful schemes for systematizing the general construction of centerings have been invented, and many of these have simplified the problem so that the main cost is in taking down and putting up the forms.

Most of these centering schemes are used for rough concrete work, where the surface is to be plastered afterward; but when finished surfaces are to be produced, the cost of centers is more than doubled. If the mechanic trained to do finished-center work be told to make rough standard centers, he will spend nearly 7½ hours carefully getting ready to do work which would take the other man ½ hour; conversely, the rapid standard-center man if put on perfect-center work, would do in ½ hour what would take the perfect-center man about 7½ hours to undo and do over again. False conceptions and misrepresentations on the part of competitors in concrete work have led them into bitter warfare by presenting owners with what they term "finished surface" free of cost. Is it not possible to conceive a method of trying uniformly rough surfaces instead of uniformly smooth surfaces, as a help toward solving the problem of centers?

The indirect method may be used to cheapen centers, that is, constructions may be used which do not require the expensive centering necessary in ordinary forms of reinforced concrete. If such constructions do not advance the total cost by increases in other directions, there will be a net saving. As is evident, the forms or centers for ordinary reinforced concrete must be sufficiently heavy to maintain a perfectly independent structure under the tendency to warp and deflect, due to the fact that they are alternately wet and dry, and also on account of the heavy load of the concrete. Where a steel skeleton (such as the columns in the McGraw Building) maintains the construction lines, one may use for centers material which is much lighter, and more easily worked and handled than with ordinary reinforced concrete.

The author does not describe the wall construction of this build-

Mr. Waite. ing, but the writer believes it to be of some form of reinforced concrete. In wall construction, the conditions are very different from either column or floor construction. In both columns and floors, concrete makes a saving in steel, but, in wall construction, this element of saving does not enter. Further, in monolithic wall construction two forms must be kept plumb, but in floors only one form is required, and gravity helps to hold this in place. The writer, with the very best of assistants, after finishing several buildings having complete concrete wall construction averaging 8 in. thick, concluded that the cost of the concrete in such wall construction was of small consequence, and could be safely neglected in totaling up the entire cost of the wall. In factory construction, where there are practically no walls except panels under windows, no such difficulties are encountered as in dead wall construction.

Nothing is stated definitely by the author concerning the character of the steel used in the floor construction, other than that round rods were used. The kind of bars and the character of the steel in them seem to command a great deal of attention at present. The only logical conclusions that can be drawn from the claims of the big grist of deformers (with increased capacities for each new deformation in their rods) is that they are developing the art toward a state where (according to claims) practically nothing but bond and grip will be required, and steel for tension, etc., as now designed, will become of little consequence.

In referring to some recent constructions which the writer has executed, the only excuse he has to offer for so doing is that such improvements have come after quite a lengthy experience in general steel and reinforced concrete construction, and, being a product of natural evolution, they belong to the general scheme of development toward something higher. The writer's experiences have been unlike those of many engaged in reinforced concrete construction, because, in most cases, he has had to contend with the conditions existing in crowded parts of large cities, where space for storing materials and performing work is extremely limited, and where great rapidity of erection is necessary; and, on account of the extra height of buildings, safe construction must be considered.

Having been fundamentally trained in steel construction, followed by the fire-proofing of the steel, and subsequently having pursued general reinforced concrete construction, the writer was forced to consider the merits and demerits of the combinations of these three factors in building construction for the conditions found in large cities.

The safety and the speed of steel construction were apparent, and the advantages in the use of concrete for the protection and fire-proofing of steel were well demonstrated. Then followed the combina-

REINFORCED CONCRETE GARAGE AT WHITESTONE, LONG ISLAND

tion of steel with concrete (formerly used for fire-proofing), and this developed into a system which possessed all the merits of the steel skeleton construction and the advantages of reinforced concrete. In this system (known as System "M," in its order with other systems) there is required only from 35 to 40% of the steel necessary for the conditions in which the steel does all the work. The concrete—which must be used for fire-proofing—is made to do the remainder. The light steel frame is run up ahead of the concrete, in the usual manner for steel frames, and is made strong enough to take all tensional and shearing stresses in the subsequently reinforced construction formed by the steel and concrete. The combination forms a truly reinforced structure. Mr. Waite.

Work can be done on several stories simultaneously, as in other steel construction. The necessary forms are simplified, as compared with those required in most reinforced concrete constructions, on account of the assistance given by the steel frame. Within the last two years, some twenty buildings in the vicinity of New York City have been constructed by this system.

From January 1st to April, 1907, while the McGraw Building was being erected, three buildings in that vicinity were constructed in which the floors were of this form of construction, namely: The Bonwit-Teller Building,* at 15 and 17 West Thirty-fourth Street, shown by Fig. 1, Plate LVIII; the Salvation Army Warehouse,† at 533 to 537 West Forty-eighth Street, shown by Fig. 2, Plate LVIII; and the Strack Building, at 214 to 220 East Twenty-third Street.

In these buildings, steel columns carry the entire loads, but, where conditions permit, a light steel frame, similar to the construction used in the McGraw Building, carries the dead floor loads.

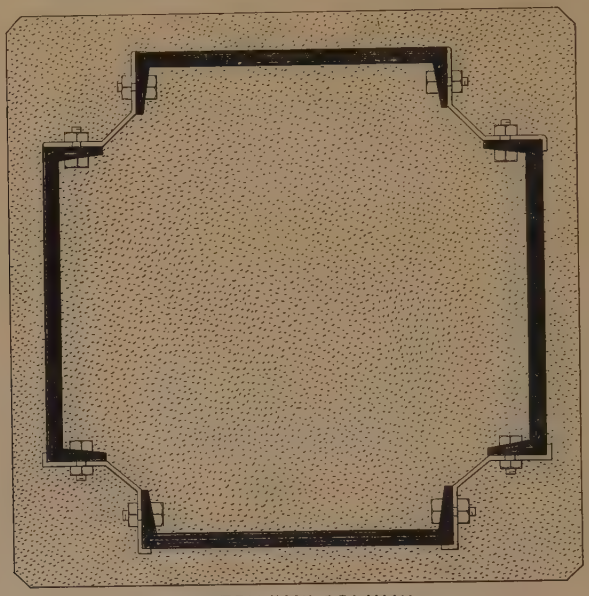
The general form of the steel in this combined column construction is shown by Fig. 1. It is made of channels or similar sections disposed centrally with respect to floor beams and girders, and the separate steel members are connected at the corners. The writer has found this to give a very simple and effective steel skeleton which, he believes, affords ample means for the proper combination of the steel and concrete.

Some months ago, in building a garage for the use of his family, at Whitestone, Long Island, the writer concluded to make the entire building of reinforced concrete in order to demonstrate the economy of a new form of construction suitable for a small number of laborers. The building is 40 ft. long and 20 ft. wide, and has two stories, and an attic. The walls consist of a series of 12 by 8-in. reinforced pilasters, spaced 5 ft. apart between centers. Between these buttresses, and erected simultaneously with them, there are concrete blocks, 3 in.

* *Engineering News*, April 25th, 1907.

† *The Engineering Record*, June 22d, 1907.

Mr. Waite. thick and 12 in. wide. In the lower story the blocks are flush with the outside of the pilasters, and in the second story they are kept back from the front to give the effect shown in the photograph, Plate LIX. The floors were reinforced with 4 by 7½-in. steel beams, resting on the pilasters, and having shear bars extending up from holes in the webs



SYSTEM "M" COLUMN

FIG. 1.

of the beams. One handy man and two laborers constructed the foundations and all the walls and floors in about 8 weeks. The walls were run up several feet above the second floor in order to make a full story of the attic.

Mr. Goodrich. E. P. GOODRICH, M. AM. SOC. C. E.—The speaker's connection with the McGraw Building, in a supervisory capacity, during the major portion of its design and construction, makes Professor Burr's description of special interest to him. In a few particulars, that description may be somewhat amplified, for the sake of noting additional points of interest.

The power plant for the building is located in a sub-basement situated in the southwest corner. Consequently, the columns in that portion of the building are somewhat longer than the others and exceed the dimensions given in the paper by 12½ ft., making the length of the longest column 172 ft. It may also be of interest to note that the "reinforcement" in the first length of one of these columns weighed 14 050 lb.

Mr. Goodrich.

The windows on the sides and rear of the building are of wire-glass in metal frames, so that practically the only possible additional device which could be added to give security against fire, would be a complete automatic sprinkler system. In consequence, the McGraw Building is one of the best in the city, as far as insurance conditions are involved, and carries a very low rate for both the building and the contents.

The column spacing was determined primarily by the dimensions of modern printing presses, nearly half a score of which are now in operation on several of the upper floors of the building. This fact brought about the use of rectangular floor bays, while a more nearly square arrangement would have been slightly more economical, had it been possible to design the building in that way.

Of the several new features in the building, of course, the column design is the most unusual. While the whole arrangement, as finally worked out, proved highly satisfactory, from a construction point of view, it may be open to some adverse criticism, from a solely economic standpoint. As shown by Mr. Douglas, a design for purely structural columns would have cost less money, and Mr. Stern suggests that, even when fire-proofed, such columns would have been smaller than those used. Plenty of evidence has been adduced from the San Francisco conflagration to show that no comparison can be made between structural columns, however well "fire-proofed" in the usual manner, and such columns as are used in the McGraw Building. No comparison is fair unless this superiority to resist fire is capitalized. On the other hand, columns of the Considère type likewise possess this good quality, their principal drawback, under such conditions, being their size. It may be of interest to state that, early in the history of the design of this building, the speaker caused to be prepared a design of a typical column, of the Considère type, based on the accepted stress requirements of the New York City, Manhattan Borough, Building Regulations at that time. The columns, of course, were circular in section, with a diameter in no case greater than the diagonal of the corresponding square column of the Burr type, finally used. The estimated cost of the Considère column, on a conservative basis, showed an apparent saving in its favor of approximately $10 000 for the whole building.

Two other small objections to the Burr column were also discovered, which were almost entirely obviated during the progress of the work, and could be entirely remedied in future designs. The wide faces of the angles in the lower stories, and the wider expanses of some of the splice-plates, made necessary a special wrapping of wire or wire lath to hold the fire-proofing concrete in place; and the extreme rigidity of the column steel, made necessary a much more careful adjustment of the forms than is usually required for reinforced concrete build-

Mr. Goodrich. ings. In most cases, the less rigid reinforcing rods are given slight eccentricities, which do not affect their efficiency seriously, and are thus made to accommodate themselves to small variations in the spacing of the forms, and thereby save some labor cost. This latter possible defect of the rigid reinforcement may even be considered a real virtue, in the eyes of some people.

The speaker is aware of really very few reliable tests of reinforced concrete columns, and of none which possessed anything like the percentage of longitudinal steel found in those of the McGraw Building. Some time ago, the speaker arranged for a series of tests on specimens designed after the Burr type, and of practically full size, but, unfortunately, the results have not yet been secured. In this, it is well to note a fact to which Professor Morsch, of Zurich, calls attention, in his "Eisenbetonbau," that the efficiency of longitudinal rod reinforcement decreases with the increase of the percentage used, at least up to 4%, and that no one knows how larger amounts will act. It is thus incumbent upon designers to exercise great care in selecting working stresses for concrete columns possessing considerable longitudinal steel, as the field is absolutely unknown at the present time, and some serious trouble may result for inexperienced designers who follow rules blindly.

Another point to be noted is the fact that most experimenters on concrete columns have concluded that the concrete appears to carry much the larger percentage of the load until it has reached a stress far above the usual allowable working one, when the steel comes into more pronounced action. Of course, this conclusion is based on computations involving an assumed modulus of elasticity of the steel and the observed stresses and strains of the column. The distribution of stress, above described, is probably due to the fact that the stress-strain curve for concrete is not a straight line, thus demonstrating the existence of a variable modulus of elasticity. From these facts, it might seem to be more rational to reverse the condition as to the dead and live load carrying capacities of the steel and concrete in the Burr column, and require the concrete, at say 750 lb. unit stress, to carry all the dead load and then add enough steel in structural form, if so desired, to carry the total or reduced live loads.

Another item of design in the McGraw Building to which special attention was paid, was the connection between the reinforcing rods and the column steel. This was worked out so effectively that the steel erectors of the columns often attached, to any convenient point of the beam reinforcement, one end of the turnbuckle which they used for plumbing the column sections. In no other reinforced concrete building within the speaker's knowledge could this be done.

With such rigidity of column steel and its firm connection with the beam rods, the best method of beam design would seem to be that

of cantilevers or continuous beams throughout, instead of simply supported members. Such an arrangement of the steel in a concrete beam has the following advantages: Mr. Goodrich.

Maximum shears occur at points where maximum moments are found, and, in consequence, where most steel is placed.

Not as much steel is found near the bottoms of beams, where it would be exposed to the most trying effects of fire.

Such a method of design obviates the tendency to sharp deflections near the supports, with the resulting probability of the occurrence of cracks at points where the shear is the greatest.

Such design gives most resistance against the type of failure observed in impact experiments.

There is also less likelihood of the displacement of reinforcement, because it is in view during the greater part of the process of concreting.

All beams and girders throughout the McGraw Building were designed as fully continuous, or restrained, even where supported in the outside columns and walls. The drawings show just as much steel over the supports as in the centers of the spans, and, since the factor used in connection with the moment at the latter point is $\frac{1}{10}$, according to the requirements of the Building Code, the factor for the supports is the same. Thus it is seen that, when compared with $\frac{1}{24}$, and $\frac{1}{12}$, the theoretically correct values, almost twice as much beam and girder reinforcement was used as theory would dictate. This extra material was used in a literal compliance with the anomalous wording of the New York Building Code. A comparison of this building with numerous others has led the speaker to the conclusion that the requirements therein contained are rarely complied with literally, and that this faulty requirement of the code has been the real cause of much poor work.

To the speaker, no reason is apparent for using, over points of support, more steel than enough to satisfy the theoretical moment formula, with a coefficient of $\frac{1}{12}$. When that amount is used in that way, only half as much, of course, is theoretically necessary in the lower part of a beam at its center, while the building requirements specify as much as would be indicated by a coefficient of $\frac{1}{10}$, which is even more than is needed in the upper part of the beam over a support.

Some slight argument may be advanced for using as much steel below as above, in the two locations, from the fact that eccentrically placed partial loads on continuous members resting on perfectly movable supports, subject the members to maximum positive and negative moments which are much larger than those produced by a continuous load, as usually considered. This fact has often led the speaker to

Mr. Goodrich. recommend a partial concession to the older ideas of design, and to use equal amounts of steel over the supports and at the centers of spans, determining this quantity by the coefficient, $\frac{1}{12}$. This distribution allows of an economical design for, and method of handling, the rods; it meets practically all the requirements of partial loads, and, at the center of the spans, uses within 20% of the quantity of steel required by the New York Building Code, with 100% better distribution, as far as prevention of cracks is concerned.

The reinforced concrete beams and girders of a monolithic concrete building are not beams and girders at all, in the sense of the wooden and steel ones in the older types of structures, which simply rest on brackets and have ample opportunity for motion in each joint. Until cracks have formed, the concrete beams are really extended brackets on the columns and other members, and should be designed as such. The early workers with reinforced concrete were influenced too largely by the old type of structure, and few designers have even yet grown into the true spirit of the newer material.

Thus it has transpired that the McGraw Building has a floor construction which is rated far below its true safe carrying capacity. Were any floor loaded to failure, the latter would probably take place by shear, or rather diagonal tension. In relation to this, however, it must be stated that the speaker never has understood why those in actual charge of the design of the reinforcement (other than the author of the paper), invariably used an odd number of rods to resist tension. By so doing it is impossible to bend upward the same number of rods at each end of a beam to assist in resisting shear, so-called. Thus, five rods might be used in a given case for tension reinforcement. Three could be bent upward at one end, but then it is practicable to bend up only two at the opposite end of the adjoining beam without causing a congestion of steel over the support. If the two bent rods were just sufficient to resist the shear, the three rods at the other end would give 50% better efficiency at that end; and 20% more resistance would have been secured at the weaker end by using six rods of smaller individual (but aggregating the same total) area, and bending up three rods at each end.

The floor forms were designed with especial care. They were collapsible in type, and were erected in an exceptionally substantial manner, so as to be capable of serving as platforms from which to erect the structural work of the columns. The determination, afterward made, to use the central tower for erection purposes, rendered this special reason for heavy forms unnecessary, but their value was repeatedly shown for other reasons, and the speaker is decidedly of the opinion that a little extra material in excess of that sometimes seen, is of real economic advantage. The forms were designed by Mr. J. G. Ellendt, and were all built in a special shop; they were hauled to the

building site on trucks, and the whole truck-load was hoisted by the central derrick in a single operation and set practically in place. The speed actually attained in erecting the building, shows how well the form work was prepared and carried on, because that work is the crucial part of the erection of all concrete work. Matched and dressed material was used throughout, always well coated with oil, so as to obviate the necessity of special surface finish if possible. However, the rapid and repeated use of this material during the winter soon disclosed the fact, the truth of which has always been held by the speaker, that it would be necessary to plaster the building, if it was to be given a character on a par with the average office structure. Mr. Goodrich.

No plans of the forms are included in the paper, although reference to them appears at one point.

In the speaker's opinion, the tower used in the erection of this building was really a factor of large economy. For instance, all concrete was hoisted to each floor in buckets dropped through the elevator shafts to the mixers, which were in the basement and placed so as to dump directly into the buckets as they rested. The booms swung the buckets so that they could be dumped exactly at the desired points, thus obviating the use of other hoists, hoppers, wheel-barrows, runways, etc. This method proved so effective that very often the cost of all labor on concrete for considerable quantities would not exceed 40 cents per cu. yd.

The speaker certainly would repeat the use of that special contrivance on a similar operation, except that he would stiffen the structure to a somewhat greater extent, and would use 12 by 12-in. timbers for corner posts instead of the 10 by 10-in. posts used in this instance. The tower structure also served as a storage space, and was of almost inestimable value in this respect, because of the congested portion of the city in which the building stands.

During cold weather, besides making use of the salamanders, as described by the author, the concrete was mixed with hot water, and all aggregates were heated so as to prevent frozen lumps from getting into the work. On one operation with which the speaker was connected he once removed a lump of frozen sand from a column in which it would have occupied about 15% of the total area. The necessity of heating the aggregate is obvious, since, even when boiling water is thrown into the mixer, it has such speed of operation that not enough time elapses to thaw frozen masses and get them properly distributed, before the mixing process is complete.

With the methods used on the McGraw Building, even in the coldest weather, the concrete would reach the point of deposit at a temperature ranging from 50 to 75° fahr., and would attain its initial set while it was still warm to the touch. The salamanders maintained a temperature in the dead air spaces between the beams, which often reached 100° fahr., and seldom fell below 60°, even in zero weather.

Mr. Goodrich. Of course, the special interest of this paper centers around the type of column. The speaker feels that, even at the present time, the designing of reinforced concrete columns is like working in a darkened room, and this is said even after personally making a large number of column tests, and after carefully analyzing nearly a hundred others. If carefully designed, and when a proper relation exists between the longitudinal and spiral steel, the speaker considers a Considère column entirely practicable for a reasonably high building with comparatively light floor loads; but, for more lofty structures, the opinion is gaining strength, in the speaker's mind, that a regular structural steel column should be used, in connection with reinforced concrete girders, beams, and floors, if desired. This structural column, however, should be of some open design, and it should be completely filled with concrete and surrounded by a fire-proofing at least 3 in. thick over all extreme edges. Such a composite building will be more economical than any other, in yearly carrying charges, including interest on first cost, insurance, maintenance, heat, etc., and a correspondingly larger income can be derived therefrom.

Messrs. Condron and Sinks. T. L. Condron and F. F. Sinks, Members, Am. Soc. C. E. (by letter).—No good reason can be offered for not exercising the same common sense in designing reinforced concrete structures as that expected and demanded in designing steel or timber structures. Too much is heard regarding "systems" of reinforced concrete, and too little regarding the simple application of the well-known laws relating to the strength of materials and the distribution of stresses in such structures. It is not many years ago that iron bridges were built according to one or another special, and generally patented, type. To-day the "patented bridges" are limited to draw bridges, which, after all, are machines as well as structures. It is doubtless true that, in a large measure, the present wide use of reinforced concrete is due to the energetic promotion of various so-called "systems," together with the equally energetic promotion of concrete construction by makers of various forms of reinforcing materials. While, in some cases, capable and conscientious engineers have done splendid work in developing new and better designs for reinforced concrete as a substitute for designs of steel or masonry structures, in other cases, less capable, and in some instances ignorant, men have produced "systems" which would be ridiculous if they were not dangerous.

In designing reinforced concrete, the writers have endeavored to follow the same methods of analysis of stresses and proportioning of parts as they use in designing steel structures. They have studied carefully all the experimental and research work done by the leading technical schools and universities, and believe that more can be gained by such study than by simply developing any refined theoretical analysis of the strength of concrete reinforced with steel.

Messrs. Condron and Sinks.

The writers present herewith illustrations of what they believe to be rational designs of reinforced concrete construction. Care has been taken to have these designs free from every unnecessary complication, the whole aim being to gain great strength and everlasting durability with the most simple construction possible.

The author's description of the McGraw Building is of especial interest, as there are several features in its design which are similar to those used by the writers; therefore, they present the following description of one building, and some notes regarding two others, designed by them.

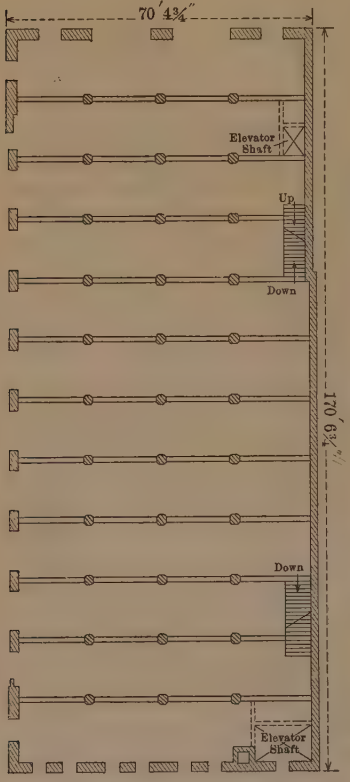
The Manufacturers' Furniture Exchange Building, in Chicago, the reinforced concrete features of which were designed by the writers, as Consulting Engineers for the Architect, Mr. William Earnest Walker, was designed in the spring of 1906 and completed near the close of that year. In the McGraw Building, as well as in the buildings designed by the writers, the columns have been reinforced with latticed steel angles. As far as the writers are aware, the first reinforced concrete building in which columns of this form were used was the Watson Building, in Chicago, built in 1905, for which Messrs. Huehl and Schmidt were the Architects. The writers' original recommendation for the columns of this building was that the angles be latticed, but they were actually built with horizontal tie-plates, as shown by Fig. 1, Plate LX. At the time this photograph was taken, the view, Fig. 2, Plate LX, was also taken on the first floor, where concreting was going on, the forms for the floors above being supported so that they did not interfere with the placing of concrete on this floor.

The general plans for the Manufacturers' Furniture Exchange Building were completed in June, 1906, and the contracts were let about July 1st. The building is near the business center of Chicago, and has a frontage of 70 ft. on Wabash Avenue, running back 170 ft. on Fourteenth Street to an alley. The general appearance of the building is shown by Fig. 1, Plate LXI. It is an eight-story and basement building, designed for furniture show rooms, warehouse purposes, or light manufacturing. The floors are designed to carry live loads of 150 lb. per sq. ft. on the lower floors, and 100 lb. per sq. ft. on the upper floors. Fig. 2 is a plan and Fig. 3 a cross-section of the building, showing the general arrangement of the columns and beams. On the first floor a bulkhead is carried around on two sides, supporting platforms for the show windows and permitting half windows for lighting the basement. The second to eighth floors, inclusive, are exactly alike. The roof is of reinforced concrete, and has two rows of saw-tooth skylights.

In designing the columns, the ratio of the moduli of elasticity of steel and concrete was assumed as 15 to 1. The columns were not

Messrs. Condron and Sinks.

considered as hooped concrete, only 500 lb. per sq. in. being allowed for the working stress on the concrete and 7 500 lb. per sq. in. on the steel. Only one change was made in the size of the concrete columns. From the basement to the third story the columns were 24 in. square, and above that they were 20 in. square. The corners of the columns were rounded to a radius of 4 in., except in the basement.



PLAN OF 2ND TO 8TH FLOOR
FIG. 2.

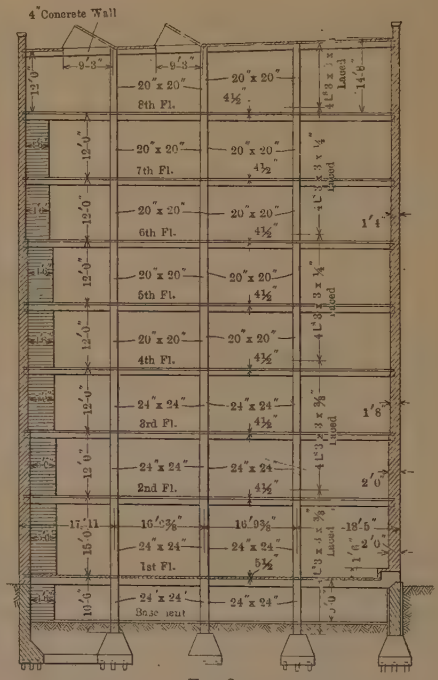


FIG. 3.

Fig. 4 shows the typical reinforcement of the columns, girders, and slabs. Temporary cross-angles were bolted to the steel column reinforcement to support the column forms, and, in turn, the floor forms above. After the concrete for one floor was finished, the weight of the form work of the floor above was supported by shores in the usual manner, resting directly on the finished concrete floor. The temporary angles were then removed from the columns, and the column boxing was closed, preparatory to casting the concrete in the column section

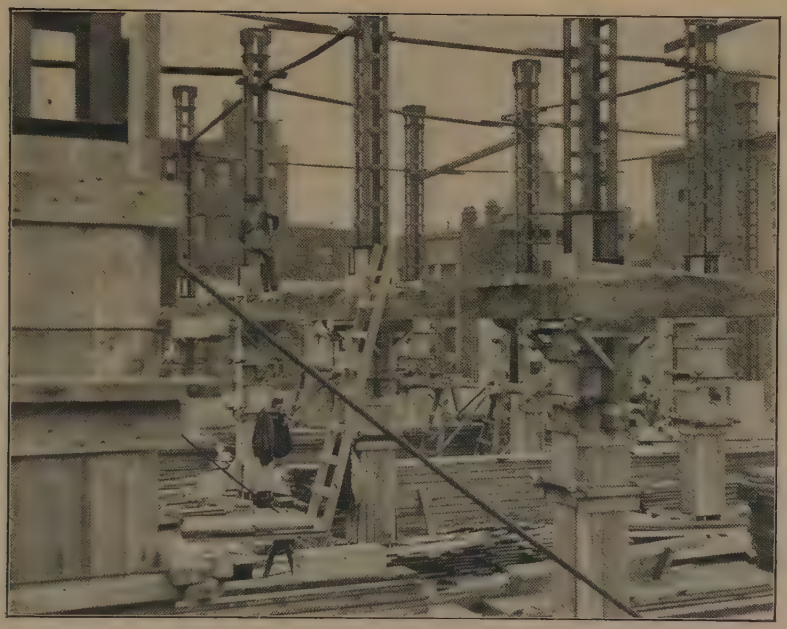


FIG. 1.—SECOND AND THIRD-STORY COLUMN REINFORCING, WATSON BUILDING.

FIG. 2.—CONCRETING ON FIRST FLOOR, WATSON BUILDING.

above the finished floor. This is all shown quite clearly in Figs. 1 and 2, Plate LXII. Messrs. Condron and Sinks.

Fig. 1, Plate LXII, is a photograph taken at the beginning of the concreting work on the first floor (October 10th, 1906), and when taken, the forms were completed for the first floor, the reinforcement of this

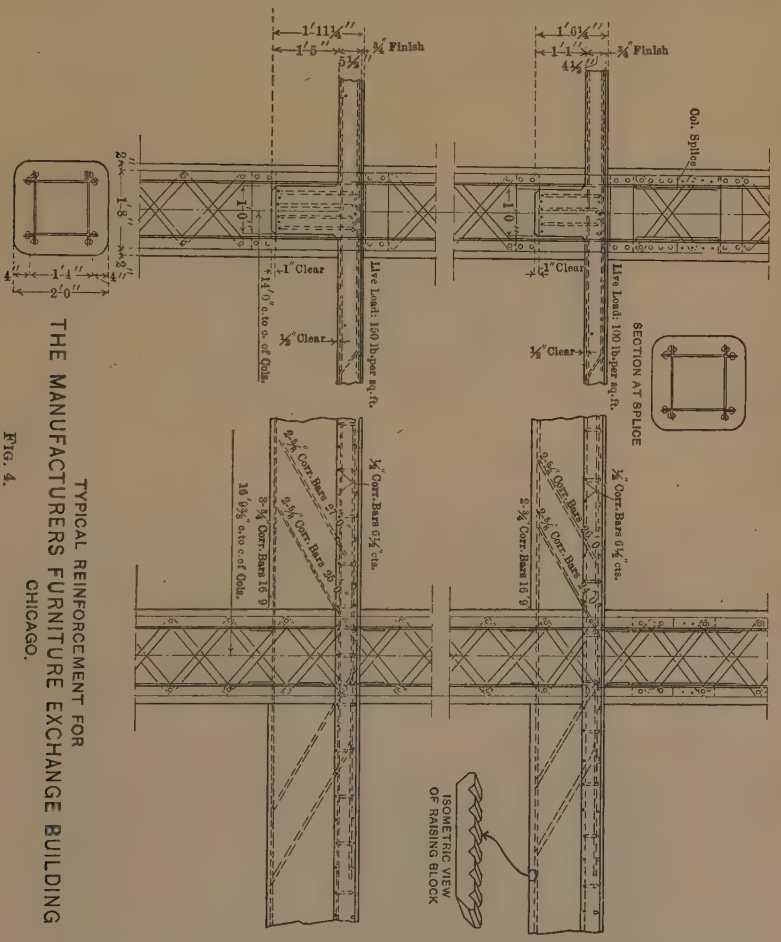


FIG. 4. TYPICAL REINFORCEMENT FOR THE MANUFACTURERS FURNITURE EXCHANGE BUILDING CHICAGO.

floor was in place, the column reinforcement, extending from the footings to the level of the second floor, was also in place, and the basement columns were cast. The character of the column reinforcement is shown very clearly in this photograph. The points where the lacing of the columns is omitted near the top are the openings left for the reinforcement for the second-floor beams to pass through.

Messrs. Condron and Sinks.

In Fig. 2, Plate LXII, the temporary angles may be seen near the bottom of the column in the foreground. This photograph was taken on November 6th, and shows the concreting in progress on the second floor. After the concrete work on the first floor was finished, no more concreting was done until after the forms for the second and third floors were both completed. The carpenters then worked on the third floor, building the forms for the fourth floor. At this stage of the work the reinforcing material for the second floor was placed, and the concreting of this floor proceeded. The photograph, Fig. 1, Plate LXIII, was taken at the same time as Fig. 2, Plate LXII, from which it will be seen that the exterior walls had been run up practically to the level of the fourth floor, and the carpenters are seen working on the fourth-floor forms, and, as stated previously, concrete was being placed on the second floor. From this time forward, both the concrete gang and the carpenters were kept constantly at work, the carpenters being two stories ahead of the concrete gang. In order to prevent freezing, the window openings were closed, and coke fires were kept burning in salamanders in the story directly under the floor on which concrete was being placed. As a consequence, the concreting went on at a temperature that was considerably above the freezing point, even in the coldest weather. The cement finish was put on the concrete floors as soon as the first concrete had taken its initial set. Owing to the prevalence of rainy weather during this construction, considerable trouble was caused by water dripping from the forms on the newly finished cement floors, and great care had to be exercised to protect these floors from injury.

Fig. 2, Plate LXI, is a photograph, taken on December 15th, when the exterior walls were finished. It shows the concrete hoist on the side of the building, with a dumping bucket just below the level of the fourth floor. The scaffold at the rear of the building carried the elevator used for raising brick. The concrete mixer is shown on the ground at the base of the concrete hoist.

As already stated, concreting on the first floor began on October 10th, and the concrete roof over the eighth story was completed on December 27th, only 66 working days intervening between the time of starting the first floor and the completion of the roof. On December 31st the tenants began moving into the practically finished building.

Fig. 2, Plate LXIII, is a photograph of the third floor, taken on December 29th, and is typical of the upper floors.

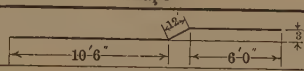
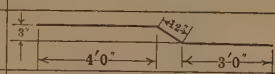
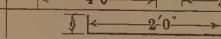
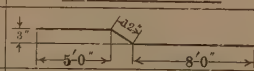
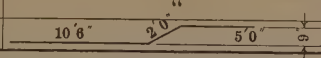
The details of the reinforced-concrete construction are shown quite clearly in Fig. 4. It will be seen, both in the beams and in the slabs, that some of the reinforcing bars run straight through on the bottom, and some are turned up to form the reinforcement for the upper face of the beams and slabs in such manner as to provide as much steel in

FIG. 1.—MANUFACTURERS' FURNITURE EXCHANGE BUILDING, CHICAGO, ILL.

FIG. 2.—MANUFACTURERS' FURNITURE EXCHANGE BUILDING, DEC. 15TH, 1906.

the top of the beams and slabs over the supports as in the bottom of the beams and slabs between the supports. Messrs. Condron and Sinks.

Fig. 5 is a bill of bars for the slabs of one floor, and Fig. 6 is the bill of bars for one girder, taken from the working plans. These bills of bars were prepared carefully, and were shown on the working plans, together with sketches of each different beam or girder, so that the contractor was able to get out the correct number of bars, and bend them to the shape required. By following the plans, it was a simple matter to select the right bars, and place them properly in the beams and panels. The specifications required that no concrete should be put in until the inspector had checked and approved the placing of the reinforcing material. The floor bars were held the proper distance

Mkd.	Total No. of Bars	Size	Length	Shape	Location
A-A	1324	½″	17′6″		P_{16} to P_{24} incl.
B-B	224	"	14′0″	Straight	$P_{19\text{-}20\text{-}22\text{-}23\text{-}24}$
C-C	9	"	6′0″	"	P_{21}
D-D	20	"	8′0″		P_{25}
E-E	235	"	2′6″		Anchors
F-F	5	"	14′0″		P_{24}
H-H	5	"	9′6″	Straight	P_{24}
X-X	224	"	23′0″	"	Longitudinal
Y-Y	18	"	18′0″	"	"
H-H	3	"	9′6″	"	"
J-J	4	"	17′6″		P_{18}

BILL OF BARS IN SLABS FOR ONE FLOOR

FIG. 5.

above the forms by 2-in. lengths of round iron of the proper diameter, while the bars in the beams were supported by two cement blocks in each beam like those shown in Fig. 4. About 1 300 of these cement blocks were required for the entire building, and each block was reinforced with two No. 8 wires, so that they could be handled safely. These blocks were found to work perfectly, the bars resting in the notches, thus being held in their proper places while the concrete was poured into the beams.

Fig. 7 shows the reinforcement of the stairways, of which there were two flights running from the basement to the eighth floor.

The extreme simplicity of the reinforcement of this building is evident, and, as corrugated bars were used, it was not necessary to make bends at the ends of the bars or use other means of insuring bond, the

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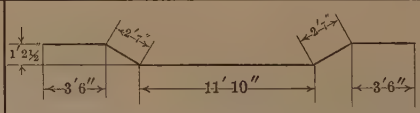
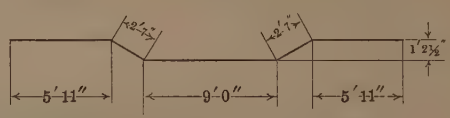
form of the bar giving in all cases an absolute bond between the concrete and the steel.

The total cost of the reinforcing bars delivered in Chicago was almost exactly 5% of the cost of the building, and, while the special bar used cost more than plain or other forms of reinforcing bars, the saving which would have been made by using a less expensive one would have been insignificant as compared with the cost of the building.

Under the Chicago Building Ordinances, it is necessary for reinforced-concrete floors to be tested with a load at least double that for which they are designed, and this ordinance requires that the floors thus tested shall show no evidence of failure and shall not deflect more than $\frac{1}{700}$ of the span, or $\frac{1}{4}$ in. for a 14-ft. span.

Mk.	No. of Beams	No. of Bars in each Beam	Shape
G_6	154	2-¾″-16′0″	Straight
		2-⅝″-24′0″	
		2-⅝″-26′0″	

BARS REQUIRED FOR BEAMS G_6

FIG. 6.

Under this ordinance, these floors were tested with a load of 350 lb. per sq. ft., covering an entire panel of 14 by 17 ft., under which test load a deflection of less than $\frac{1}{16}$ in. was measured.

Later, the writers followed this method of design for the warehouse of the Advance Thresher Company, at Kansas City (Mr. J. C. Llewellyn, Architect), the typical reinforcement of the columns, girders, beams, and floor slabs of which is shown in Fig. 8. These floors were designed to carry a working load of 250 lb. per sq. ft. in addition to the dead load. In this case, floor slabs of 8 ft. span were adopted, and these were carried by concrete joists framing into concrete girders. The spacing of the columns was very irregular in the building, but the maximum spans were 24 ft. The type of column reinforcement was about the same as in the Manufacturers' Furni-

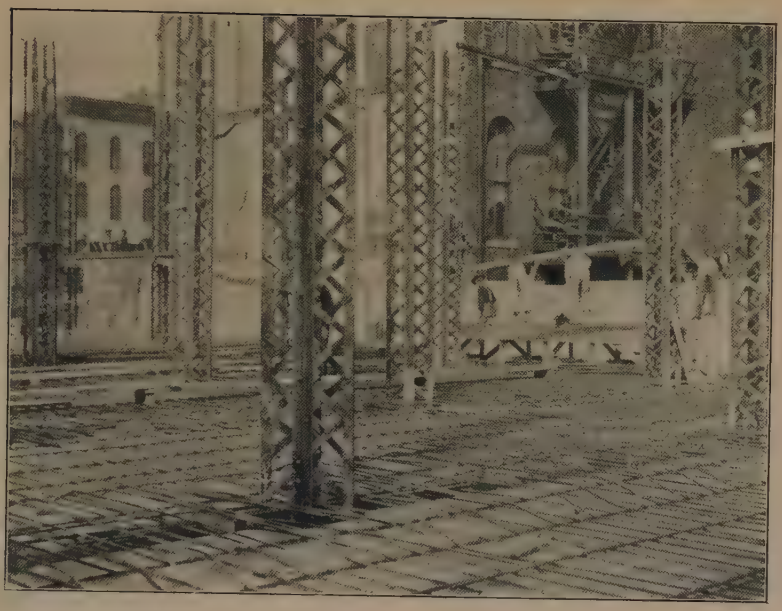


FIG. 1.—REINFORCING OF COLUMNS AND FLOOR, MANUFACTURERS' FURNITURE EXCHANGE BUILDING.

FIG. 2.—CONCRETING ON SECOND FLOOR, MANUFACTURERS' FURNITURE EXCHANGE BUILDING.

ture Exchange Building, except that the angles for this building were placed with the corners out, instead of in. Messrs. Condron and Sinks.

Owing to the fact that this building is to be used for exceedingly heavy concentrated loads, that is, the weight of traction engines having 12 000 lb. concentration on a wheel, the owners desired to satisfy them-

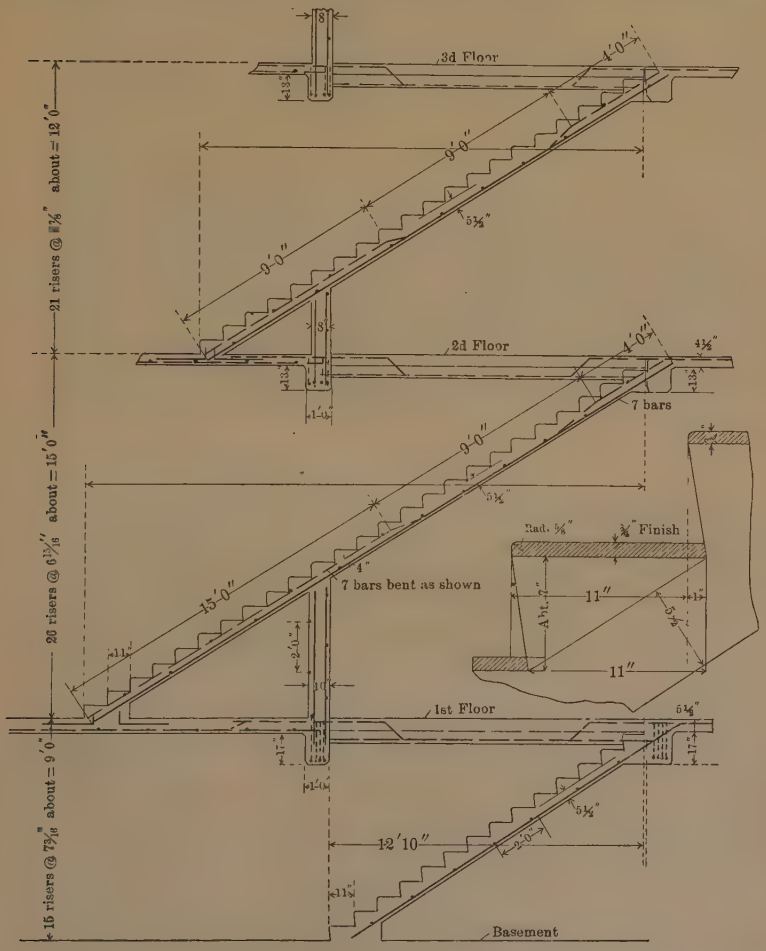


FIG. 7.

selves of the effect of concentrated loads on these floors, and, therefore, the following test was made:

Two strips of 2 by 4-in. wood, each 4 ft. long, were laid along the centers of two adjoining 8-ft. slab spans parallel with the supporting joists. On these two strips rested a platform on which a load of

Messrs. Condron and Sinks.

40 850 lb. was placed, giving a concentration of 20 425 lb. on each strip, or a concentrated load, at the center of each of these two slabs, of 5 106 lb. per lin. ft., thus producing the same moment in the slab as a uniformly distributed load over a 4-ft. width of the two panels of 1 274 lb. per sq. ft. Under this test a deflection of $\frac{1}{16}$ in. was measured. Of course, the entire floor assisted in carrying such a test

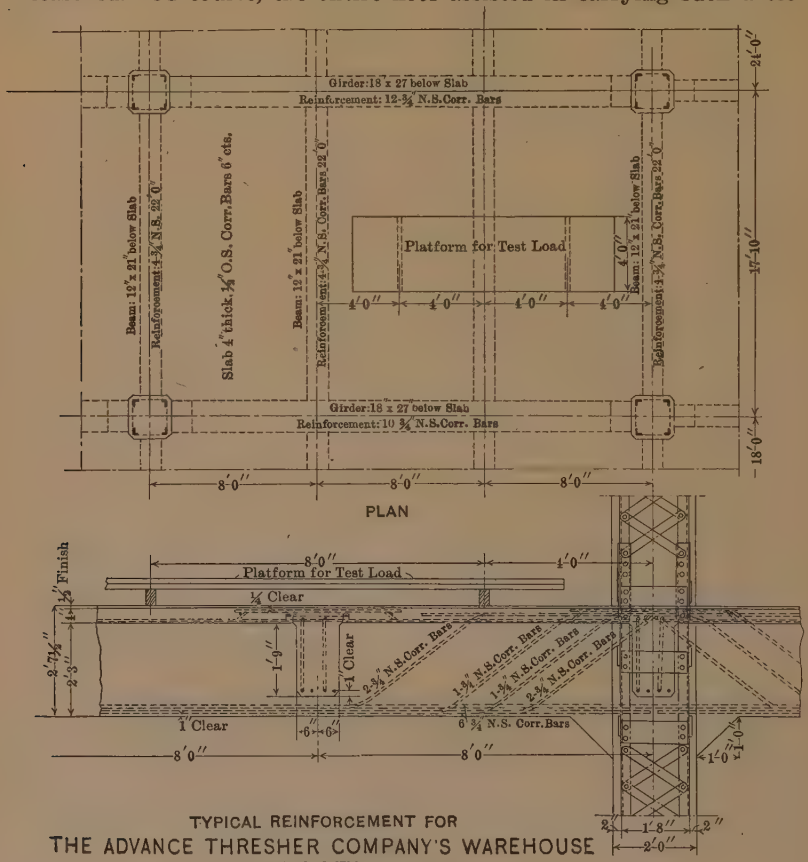


FIG. 8.

load, and this is only mentioned as illustrative of the remarkable carrying capacity of such floor slabs. Notwithstanding the fact that such floor tests give astonishing results, the writers believe that floors should be designed, not on the basis of such tests, but in accordance with conservative practice, and in all their work they have considered that, for floor slabs and intermediate beams, where the reinforcement passes over the top of the slabs and beams at the ends and well into the next

FIG. 1.—EXTERIOR OF MANUFACTURERS' FURNITURE EXCHANGE BUILDING, NOV. 6TH, 1906.

FIG. 2.—TYPICAL INTERIOR, SECOND TO SEVENTH STORIES, MANUFACTURERS' FURNITURE EXCHANGE BUILDING.

Messrs. Condron and Sinks.

panel, the moment is equal to $w\ l^2 \div 12$, and that, for end beams and end panels, where the reinforcement can only pass in this manner over the top at one support, $w\ l^2 \div 10$. The writers consider that the dead load plus the assumed live load will stress the reinforcing material to one-third of its elastic limit, and they use from 0.8 of 1% to 1% of steel reinforcement having an elastic limit of 50 000 lb. per sq. in. They have not calculated the beams as T-beams, even in such floor construction as illustrated, but have considered them as rectangular beams with a depth equal to the distance from the top of the floor to the bottom of the beam, and in them have used reinforcement as great as 1½%, but usually it does not exceed 1¼%, of the area of the beam, not including any portion of the floor slab except that which is a part of the beam section.

In designing beams and girders, the writers have used the empirical formula, $M = (450\ P + 55)\ bd^2$. This formula was adopted as a result of the study of all the tests of reinforced concrete beams which had been made in the various engineering laboratories of the technical schools prior to June, 1905. Up to that time, 202 beam tests had been reported, of which 72 were of beams reinforced with Johnson bars having an elastic limit of about 50 000 lb. per sq. in. Of these 72 tests, 80% showed ultimate strengths exceeding that given by the formula, while only 8% developed less than 90% of the formula strength, and the lowest test developed 78 per cent. This formula is only used where P, the percentage of reinforcement, is more than half of 1%, and not more than 1½%, and where steel having an elastic limit of 50 000 lb. per sq. in. is used, and with a positive mechanical bond. For 1% of reinforcement, the ultimate moment, $M = 505\ bd^2$, and, in general, that percentage has been used. The writers have considered that three times the dead-load moment plus three times the live-load moment is equal to the ultimate moment. Therefore, if the dead load is equal to one-half of the live load, it would require, theoretically, an application of four times the working live load to reach the ultimate load. The tests made indicate that this practice is on the safe side.

Mr. Stern.

E. W. Stern, M. Am. Soc. C. E.—The author states that, in view of the uses to which the McGraw Building was to be devoted, it was imperatively necessary that it should be designed to afford the greatest possible resistance to the vibration of heavy machinery. Now, is enough known about the action of reinforced concrete under vibratory loads to make certain its suitability for this purpose?

In reinforced concrete buildings, cracks occur when there are practically no vibratory loads; under the influence of vibrations continued for a number of years, due to running machinery in the building, is it certain that cracks will not develop, and that the reinforcing rods will not work loose in the concrete?

Mr. Stern. Considering the design of the columns, the author assumes that stress is transmitted into the concrete filling through the rivet heads and lattice bars of these columns, so that both steel and concrete act together as a unit. The speaker cannot accept this assumption. It seems to him far-fetched and entirely problematical. If there are any experiments to fortify the contention of the author, it would be of benefit to this discussion to have these results.

The author likewise adopts a working stress of 750 lb. per sq. in. on the concrete filling of the columns, equivalent to 45 tons per sq. ft. Such a very high unit stress is so much more than has been considered good practice (being more than double that allowed in the Building Code of Manhattan), that the author should give the Profession the benefit of the experiments upon which he bases his conclusions.

It is not clear to the writer, in examining the details, how the splices of the columns were arranged at the various joints to take care of the reduction in dimensions. For instance, the columns in the ninth story are 17 by 17 in., back to back of angles, whereas the next section of columns, supported on these, is decreased suddenly to 10 by 10 in., back to back of angles. It would be of interest to know how this change in size was taken care of in the details.

The speaker believes the type of column used to be neither as economical nor as efficient as a box steel column made of plates and channels. A column of this type, to take the same load, would be made of 15-in. channels with 17-in. cover-plates, and would build up about 21 in. square, if surrounded with 2 in. of fire-proof covering. The columns in the McGraw Building are 29 in. square in the basement and first floor, or about 40% larger in outside dimensions, and occupy nearly twice as much space; in fact, in the lower stories, these columns are actually about as large in outside dimensions as the steel columns in the thirty-two story City Investing Building, at Broadway and Cortlandt Street, designed by the firm of which the speaker is a member.

There is actually more steel in the type of column adopted than there would be in the channel and plate box column above mentioned, assuming that the entire load were carried by it, without any regard to concrete filling; and, if it were intended to fill these columns solid with concrete and surround them with a fire-proof covering of that material, there would be less concrete used, so that the column adopted was extravagant both in material and in space occupied.

The author claims that the use of the type of steel column adopted was a great convenience in erection, as it enabled the steelwork to be erected ahead of the concrete work, and afforded convenient supporting members for the adjoining forms or for other erection work. This argument would be equally applicable to the more economical type of column suggested by the speaker.

Mr. Stern.

The columns are spaced too closely together for a building adapted to loft purposes. The interior columns, parallel to 39th Street, are 15 ft. 9 in. to 14 ft. 8 in. apart. A better arrangement would have been to have these columns spaced about 18 ft. apart.

The author also states that the construction of the concrete work in this building during the winter of 1906-07 was entirely successful, thus demonstrating that reinforced concrete work may be conducted during a New York winter without material interruption. He states that this was accomplished by covering window openings with canvas, using salamanders, and covering the fresh concrete, as fast as poured, with tarpaulins or hay, or both. Now, while it may be possible to obtain first-class concrete work in this way, it is undoubtedly a great expense thus to do the work, and likewise risky, as the work may freeze at any time, especially the thin floor slabs, from underneath.

The speaker knows of a number of cases of collapse due to this cause, and he believes that the erection of reinforced concrete work, in which there are thin floor slabs, undertaken during freezing weather, carries with it grave responsibility and uncertainty.

It might be interesting to compare the quantity of steel required in reinforcing the concrete work of the McGraw Building and in that of a steel skeleton structure. A complete steel frame structure for the McGraw Building, computed for the same loads that were used by the author, would weigh approximately 1 500 tons. In the McGraw Building the steel columns weigh 655 tons, the reinforcing rods 507, making a total of 1 162 tons, equivalent to a saving in the McGraw type of 340 tons in the steelwork. This would amount to about $21 000, assuming the price of steel to be about $62 a ton. This difference, however, would most likely be more than offset in other ways in the steel skeleton type, as there would be much less concrete required, and the erection methods would be less expensive. Perhaps the author made comparisons as to the cost of these different types of construction; if so, it would be interesting to have his figures.

In an experience covering more than seventeen years in the construction of buildings, the speaker has had to deal with practically all kinds of materials, and has had charge of a number of reinforced concrete structures. In his opinion, nothing, thus far, has been devised which is comparable to the modern steel skeleton type of construction for high buildings, not only for safety, but for economy, speed in construction, and ability to make the frame as thoroughly fire-resisting as possible.

Every condition of loading can be intelligently taken care of in a steel structure, the stress in each member of the frame being capable of complete analysis, and the knowledge at hand to-day as to what the unit stresses should be has been so thoroughly tried out that it is safe to say there is practically no element of uncertainty in the design

Mr. Stern. of a steel building. In a reinforced concrete building, however, the case is otherwise. The factor of ignorance is much greater. Most of the work is done on the premises by labor more or less unskilled.

The supervision of the work during construction is of the most exacting nature, and requires high intelligence and unremitting vigilance. The difficulty of getting good workmanship, and of making the construction correspond with the plans, is very great, and, finally, after the work is finished, grave defects of workmanship may exist in spite of all the care exercised.

Mr. Mensch. L. J. MENSCH, M. AM. SOC. C. E. (by letter).—This paper has been read with great interest by the writer, and, while he does not doubt that the owners are more than pleased with the strength of the building, he has to take exception to many statements made.

The structure cannot be called a true reinforced concrete building, the columns being of steel, fire-proofed by concrete, although ostentatiously calculated as reinforced concrete columns. Neither is it the latest type of reinforced concrete building construction; it is, in fact, the oldest type of high building construction in which reinforced concrete was used. After the introduction of reinforced girder and slab construction, many years elapsed before owners and architects could be persuaded to allow the use of reinforced concrete columns, and, in most cases, latticed steel columns, fire-proofed by concrete, were used. Of the numerous buildings of this type, the writer will mention only the ten-story Audit Office of the French Government at the Cours de la Reine, Paris. The statement, that the McGraw Building is higher than heretofore considered practicable, must be contradicted. The height of the Ingalls Building, in Cincinnati, is about 220 ft. above the basement, and the height of the Pugh Power Building, in the same city, designed by the writer, is about 180 ft. above the basement. The latter is used for the same purpose as the McGraw Building, and has also the same spacing of columns; and the first section, 70 by 335 ft., proved such a success that the owner built an addition to it, making it now about 150 by 335 ft. and ten stories high. Mr. Douglas has shown clearly the waste of steel in the columns.

It is true that very few tests of structural steel columns strengthened by concrete filling have been made, and the writer is pleased to be able to mention at least one test which was made by Dr. F. von Emperger, and published in the July number of *Beton und Eisen*, 1907. Two I-beams, about $5\frac{1}{2}$ in. deep and $6\frac{3}{8}$ in. from center to center, were connected by eight flat irons $2\frac{1}{2}$ by $\frac{1}{4}$ in. in a length of 13 ft. This column failed at 100 000 lb., the I-beams buckling separately. The column was straightened out, and the space between the I-beams filled with concrete, and tested after six weeks. The composite column failed at 265 000 lb. The iron section contained 6.3 sq. in., the concrete section contained $31\frac{1}{2}$ sq. in., the radius of gyration of the two

I-beams was 2.6 in., and, from this, Dr. von Emperger demonstrates that the carrying capacity of this column is to be considered as the sum of the carrying capacities of the iron and of the concrete. He also mentions that the concrete completely separated from the iron, and crushed into pieces from 1½ to 3 ft. long. Test cubes, cut from such pieces, gave an ultimate resistance of 1 120 lb. per sq. in. Mr. Mensch.

Although this test may not be entirely convincing, it shows that, in such a column, the highest working stress on the steel section may be allowed safely, disregarding the concrete, which may be considered as acting only as a stiffener. From this it follows that it would have been safe to reduce the size of the columns of the McGraw Building.

The author is correct in his statement that the form work represents the most difficult part of reinforced concrete construction; and the success of a contractor, and also the speed of erection, depend entirely on his ability to organize his carpenter force, and to give his foremen complete working drawings, omitting not the smallest detail, even specifying the number and kind of nails; in fact, do the work on the same basis as structural steelwork. But it is also the duty of the engineer to design the building so that the form work is reduced to a minimum. For example, lumber comes only in certain sizes—a so-called 2 by 10-in. plank, is generally only 1⅝ by 9½ in.—and, if a column 20 in. square is specified, the forms can only be made by ripping the planks. On the other hand, it will be found that a column 19¼ by 19½ in. can be formed in by using commercial lumber, and it is absolutely necessary that the designing engineer should know the commercial sizes of lumber, as they vary with different localities. The same applies to girders and beams, which, as a rule, cannot be obtained in even dimensions without considerable waste of labor and material.

The use of brackets should be carefully considered. It seems that in most cases they are adopted for good luck, with no regard to statical considerations. The writer has seen many brackets, the under side of which formed an angle of 60° and more with the horizontal, which were generally not more than 8 or 12 in. in length. Such brackets add greatly to the cost, but very little to the strength, of the structure. A little consideration would show that it would be cheaper to use deeper girders and omit the brackets. A bracket is of importance only in case the underside forms an angle of not more than 25° with the horizontal.

The layout of girders and beams should be made as simple as possible. The writer cannot say that the distribution of girders and beams in the McGraw Building is the most economical, or the most favorable, for the form work. In the Pugh Building, the girders were adopted in the direction of the 21-ft. spans, and the beams in the direction of the 14-ft. spans, and were spaced about 11 ft. apart. This

Mr. Mensch. reduced the number of beams, and made the centering of the girders much simpler, and the little excess of concrete used in the slabs, which were reinforced in both directions—a necessity in every good design—was more than counterbalanced by the saving in the form work and time. The fact is that, although the floor area of this building was more than 20 000 sq. ft., and all the walls and main partitions were also of reinforced concrete above the third floor, the rate of progress was a story every 16 days, with a comparatively small gang of men.

In regard to the use of a derrick tower with four swinging booms, the writer's experience proves that the cost of the handling of the concrete and the installation of such an outfit is considerably more expensive than the use of a small elevator and concrete bucket, which empties into a hopper and is hauled in two-wheeled buggies to the place where needed.

Mr. Stevens. P. E. Stevens, Assoc. M. Am. Soc. C. E. (by letter).—In the design of the McGraw Building, careful attention has been given to those details intended to secure continuity in the beams, even though the New York Building Code does not permit the designer to take full advantage of the increased strength resulting from such continuity. This feature of the design has been made the subject of some adverse criticism, on account of the abundant reinforcement provided. The writer believes that these criticisms are not well founded. The description and drawings of the reinforcing frames for the girders show the steel reinforcement over the supports to be the same as that at the center of the span, and this has been regarded by some as a waste of material. The usual formulas for stresses in continuous beams apply in the case of concrete only when such reinforcement is provided. In the derivation of such formulas, three conditions are imposed:

1.—Unyielding supports, conforming to the unstrained outline of the beam—usually styled supports all on a level;
2.—Spans all equal;
3.—Uniform moment of inertia throughout the length of the beam.

The first of these conditions it is impossible to fulfill, the second seldom prevails, and the third is commonly ignored. This is not intended for cynicism, but is a simple statement of fact. Imperfect workmanship, uneven shrinkage of concrete, and elasticity in the material, make any assumption of "supports on a level" untenable. This alone is sufficient reason for placing small dependence on the increased strength due to continuity, when designing the beam.

An example of a very common and erroneous interpretation of the third condition is found in another discussion of this subject. Mr.

Noble has said that the formulas for stresses in continuous beams apply only when the bending moments are Mr. Stevens.

"adequately met by moments of resistance, and then only when the unit stresses in the material furnishing this amount of resistance are the same at the center span and the points of support."

This is decidedly at variance with the third condition imposed by the formulas. Uniformity of stress is far from identical with uniformity of moment of inertia.

The amount of the bending moment over the support added to that at the center of the span will give a sum equal to $\frac{w\,l^2}{8}$ in an infinite series of uniform spans uniformly loaded. If, further, the moments of inertia of the sections of the beam are the same throughout its length, then, and then only, the bending moment over the support is $\frac{2}{3}\left(\frac{w\,l^2}{8}\right)$, and that at the center is $\frac{1}{3}\left(\frac{w\,l^2}{8}\right)$. In a series of eight or more spans complying with the three conditions before mentioned, and uniformly loaded, the span at the middle would have approximately—within 1%—the above distribution of bending moments.

A design which assumes some fraction of this total, $\frac{w\,l^2}{8}$, as the moment at the support, and the remainder as the moment at the center of the span, simply because moments of resistance have arbitrarily been provided at those points to resist such moments, cannot be justified by any sound theory.

In order to show how erroneous any conclusions drawn from such assumptions may be, the writer has derived the correct moments at the supports and center of the span corresponding with various ratios between the moment of inertia for the cantilever portion and that for the suspended portion of the span.

The beam to be considered will be assumed to be one of an infinite series of uniform spans, uniformly loaded—approximated by a span at the middle of a series of eight or more spans—and with supports "all on a level." In Fig. 9 let the following nomenclature and conditions govern:

$A\ E$ = the undeformed neutral axis of the beam, with supports at A and E;

$A\ B\ C\ D\ E$ = the deformed neutral axis;

B and D = the points of contraflexure;

w = the uniform load per unit length;

I = the moment of inertia of any section of $A\ B$, or $D\ E$;

K = the moment of inertia of any section of $B\ C\ D$.

Mr. Stevens. All conditions are symmetrical, therefore the elastic curve will be symmetrical.

a = the distance, $A\ B$, or $D\ E$;

b = the distance, $B\ D$;

M_0 = the bending moment at A or E;

M_1 = the bending moment at the center of the span, C;

$B\ D$ will be parallel with $A\ E$.

$B\ F$ is tangent to $A\ B$ at B, making the angle, α, with $A\ E$ and $B\ D$;

$B\ G$ is tangent to $B\ C\ D$ at B, making the angle, β, with $B\ D$;

n and m are current co-ordinates of points in $B\ C\ D$, referred to B;

x and y are current co-ordinates of points in $A\ B$, referred to B.

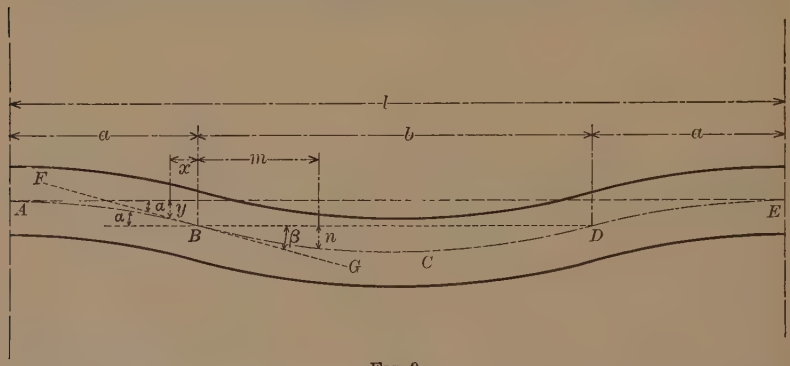


FIG. 9.

Since B and D are points of contraflexure, and bending moments at these points are zero, the beam may be regarded as made of three beams: two cantilevers, $A\ B$ and $E\ D$, and a simple span, $B\ D$. Consider first the span, $B\ D$, loaded with w per unit of length. By the well-known theory of flexure,

$$n = \frac{w\ m}{24\ E\ K}(b^3 - 2\ b\ m^2 + m^3) \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Differentiate and obtain the first derivative,

$$\frac{d\ n}{d\ m} = \frac{w\ (b^3 - 6\ b\ m^2 + 4\ m^3)}{24\ E\ K} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Make $m = 0$, then,

$$\tan.\ \beta = \frac{w\ b^3}{24\ E\ K} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Consider now the cantilever, $A\ B$: It is loaded with the uniform load, w, per unit length and the end reaction from $B\ D$ at B. This end reaction is equal to $\frac{w\ b}{2}$. From uniform load:

$$\text{deflection} = \frac{w}{42\,E\,I}(x^4 - 4\,a^3\,x + 3\,a^4) \quad \ldots\ldots(4)$$ Mr. Stevens.

and, from load, $\frac{w\,b}{2}$ at B;

$$\text{deflection} = \frac{\frac{w\,b}{2}}{6\,E\,I}(2\,a^3 - 3\,a^2\,x + x^3) \quad \ldots\ldots(5)$$

and y, the sum of these deflections, is

$$y = \frac{2\,w\,b\,(2\,a^3 - 3\,a^2\,x + x^3) + w\,(x^4 - 4\,a^3\,x + 3\,a^4)}{24\,E\,I} \quad \ldots(6)$$

obtain the first derivative:

$$\frac{d\,y}{d\,x} = \frac{2\,w\,b\,(3\,x^2 - 3\,a^2) + w\,(4\,x^3 - 4\,a^3)}{24\,E\,I} \quad \ldots\ldots(7)$$

make $x = 0$, then,

$$\text{tan. } \alpha = -\frac{6\,w\,b\,a^2 - 4\,w\,a^3}{24\,E\,I} \quad \ldots\ldots(8)$$

Since B is a point of contraflexure, $\alpha = \beta$, and

$$\text{tan. } \alpha = \text{tan. } \beta \quad \ldots\ldots(9)$$

and, from Equations 3 and 8,

$$\frac{w\,b^3}{24\,E\,K} = -\frac{6\,w\,b\,a^2 - 4\,w\,a^3}{24\,E\,I} \quad \ldots\ldots(10)$$

$$\text{whence, } \frac{b^3}{6\,b\,a^2 + 4\,a^3} = -\frac{K}{I} \quad \ldots\ldots(11)$$

The negative sign governing the second term of this equation is due to the fact that the moments in the cantilever and those in the suspended portion are of opposite sign. It will be dropped hereafter, as it is immaterial to this discussion.

By assigning to a and b consistent values, fractions of the total span, $A\,D = l$, corresponding values of $\frac{K}{I}$ may be obtained. Such values have been platted and a curve drawn through them (Fig. 10). A curve has also been drawn for the values of M_o in terms of $\frac{w\,l^2}{8}$, and one for the ratio, $\frac{M_1}{M_o}$, for comparison with $\frac{K}{I}$. Attention is called to the values of the various functions given by these curves corresponding with $a = l\,(1 - \frac{1}{3}\sqrt{3}) = 0.211\,l$. The curves show $\frac{K}{I} = 1$; hence

$$K = I,$$

$$M_o = 0.66\left(\frac{w\,l^2}{8}\right)$$

$$M_1 = \frac{1}{2}\,M_o$$

which values correspond exactly with the formula for beams with uniform moment of inertia. It will be noted that the curves for $\frac{K}{I}$

Mr. Stevens.

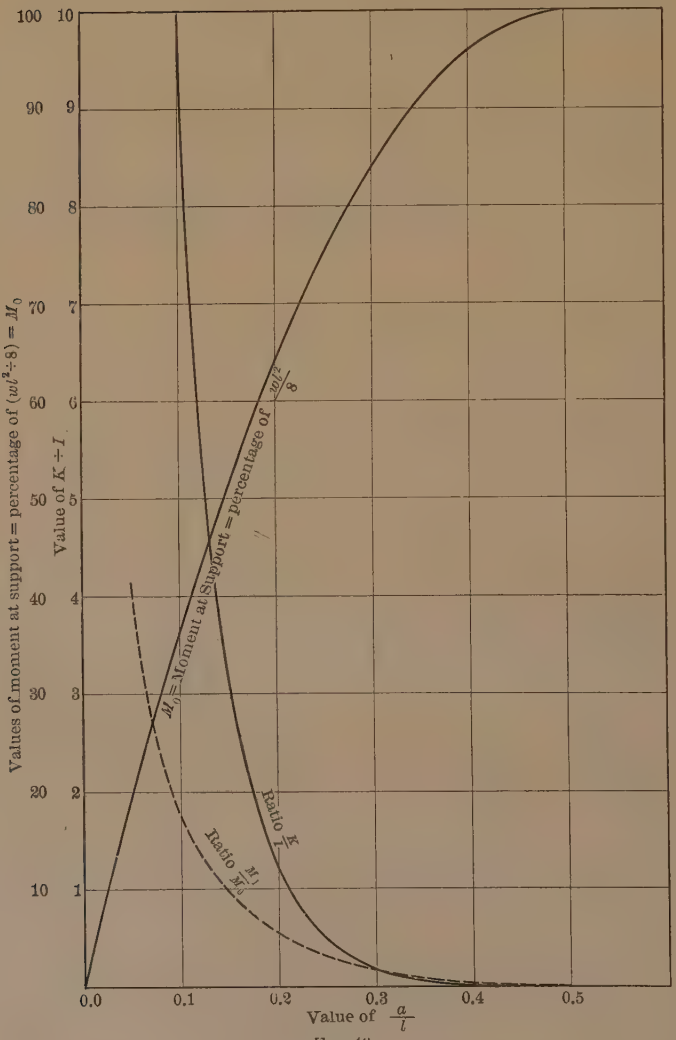


FIG. 10.

and $\frac{M_1}{M_o}$ intersect at a common value of 0.17, indicating that for these values, if the depths are uniform, the stresses at the center and at the supports will be equal. If, as is often done, $\frac{K}{I}$ is made equal to $\frac{1}{2}$ and a bending moment of $\frac{2}{3}\left(\frac{w\,l^2}{8}\right)$ is provided for at the supports, an overstress of 12½% will result. If, as is said to be the French practice, $\frac{4}{5}\left(\frac{w\,l^2}{8}\right)$ is provided for at the center of the span, and $\frac{1}{5}\left(\frac{w\,l^2}{8}\right)$ is provided for at the supports, then $\frac{K}{I} = 4$, and $M_o = 0.475\left(\frac{w\,l^2}{8}\right)$. Therefore, if the conditions of continuity and loading assumed are realized, the result will be an actual theoretical stress 2⅜ times as great as that calculated. This appears to the writer to be extremely bad designing. Mr. Stevens.

It must be borne in mind that the live load to be sustained by the beams and columns of a building is not a uniformly distributed one, but a constantly varying set of unequal loads, sometimes concentrated and sometimes distributed over varying areas. In a steel structure, where each beam is an independent span, a uniform live load, with a proper "scaling down formula" can be specified, which will enable the designer to work very close to actual conditions. In a reinforced concrete structure, where the floor slabs, the floor beams, and the girders are built as continuous beams, the stress in every beam, girder, and column is influenced by every live-load concentration, whether it is assumed so or not. In such a case, it is manifestly impossible to make close calculations, and the most conservative estimate must be placed upon the value of continuity as a factor in saving material. Also, if concentrated loads are to be provided for, the reinforcement over the supports must be made equal to that at the center of the span, as has been done in the design of the McGraw Building.

In the design of steel structures, experience has taught that forms in which the stresses in each member can be determined accurately are preferable. Many of the statically indeterminate forms, such as multiple intersection trusses and continuous girders, have almost entirely disappeared.

The workers in the younger art of reinforced concrete would do well to give most respectful consideration to this idea of using statically determinate forms, which has become so general in the design of structures in steel—a material the properties of which may be far more accurately determined or controlled than those of concrete.

William H. Burr, M. Am. Soc. C. E. (by letter).—The discussion of this paper has developed expressions of widely varying opinions, and it has disclosed some unexpected sources of criticism. Instances Mr. Burr.

Mr. Burr. of bad design, worse fabrication, and marked unfamiliarity with concrete work have been exhibited to illustrate the difficulties attending the construction of reinforced concrete work and its alleged uncertain character. It is probably fully understood among experienced and well-informed engineers, as has been often stated, that the degree of ability necessary in the design of a reinforced concrete structure, the thoroughness of inspection, and the care in fabrication are neither less nor more than required in the best quality of structural steel work. In fact, in these respects, it may be reasonably maintained that both classes of construction are in the same category. To cite badly-designed and badly-handled concrete work as an argument against first-class reinforced concrete construction is precisely like citing unscrupulously-designed and badly-built, back-country, highway bridges of poor steel as a legitimate argument against first-class structural steel work. The proper question is what can be done with competent design and first-class materials and work, not what defects can be developed by bad material and inefficiency of handling.

It has been alleged in this discussion by some, on the one hand, that the concrete filling of the columns in the McGraw Building will shrink away from the steelwork; and, on the other hand, by others, that the steelwork is not stiff enough to resist the pressure of the lateral expansion of the concrete under loading. It is probably best to allow these two opposite criticisms to annul each other.

It may be observed, however, that, if there really is that amount of shrinkage, the exterior shell of concrete encasing the steelwork of the columns would tend to draw that steelwork against the concrete filling and undoubtedly crack itself longitudinally in the effort. As no such results, to any extent whatever, have been observed, the alleged condition of shrinkage cannot exist to any discoverable extent. This apprehension of excessive contraction probably arises, in some quarters at least, from the views expressed in the discussion that the setting of the concrete is a "drying" or "drying out" of the material. Such erroneous views as to the real operation of setting might easily lead to apprehension of abnormal shrinkage. As a matter of fact, the fresh concrete in the interior of the columns was put in place in such a wet, semi-plastic condition that most of it took its initial set under a pressure of from 500 to 1 200 lb. (or more) per sq. ft. The fresh concrete was thus pressed outward with great force against the steel and the timber forms. The writer has seen the concrete cut away from steel reinforcement in many cases, including the columns and other steel reinforcement used in the McGraw Building, without, in a single instance under his observation, disclosing any tendency whatever of the concrete to part from the steel enclosed by it. There are undoubtedly such instances, where work has not been carefully done, or possibly from some special conditions where there has been negligence,

but it is believed, from extended experience in a wide range of this class of work, that the danger of defects arising from this alleged shrinkage is essentially imaginary, and that it may be ignored wherever work is properly done. Mr. Burr.

Exception has been taken to the statement that the concrete filling of this type of column is banded in the most effective manner possible. While such an expression may be interpreted to mean too much, it is believed that the statement is justified. Its meaning is that, in this type of column, not alone in this special instance of the type, the steel reinforcement, while stiffly embracing the concrete, is in a self-sustaining and load-carrying form. It is an element of strength to the column, not only by being directly load-carrying, but as acting to prevent lateral strains of the concrete, that is, lateral enlargement of the concrete under loading. Furthermore, it is such a type of reinforcement as to act initially with the concrete; in other words, both concrete and steel act together from the beginning of the loading to its full value. In the banded type of column, on the contrary, with longitudinal rods incapable of acting as load carriers in themselves, these highly desirable conditions do not exist. Professor Talbot's experiments show conclusively that the steel banding does not act to a sensible extent until the concrete has been stressed nearly up to its ultimate capacity as plain material, while the longitudinal rods are in themselves elements of actual weakness. It would seem, therefore, that the criticized expression is reasonably justified, even though the lattice bars are not always very stiff members under flexure.

It has been objected that the compression of the steel angles of these columns under loading will result in the lattice bars forcing them further apart, thus possibly breaking the bond between the angles and the enclosed concrete. At first sight this seems plausible, but, aside from the obvious fact that any such motion must be exceedingly small, if it existed to any such extent as alleged the exterior fire-proofing shell of concrete would be cracked longitudinally. The fact that no such longitudinal cracking exists conclusively disposes of the objection. This objection is also disposed of by another consideration. When a built-up steel column is subjected to loading, its tendency is to bend slightly as a whole, resulting in its parts taking very slight but concentric curvatures. Under such circumstances the longitudinal elements of the column do not tend to separate, and the lattice bars are frequently subjected to heavy compression. These results are commonly observed in full-sized column testing.

It has been stated that, under a compressive loading of 750 lb. per sq. in., the lateral strains of concrete are not more than 20% of the direct or longitudinal strains of compression, while, as a matter of fact, tests have shown that it is more likely to be nearer twice that amount.

Mr. Burr. Much apprehension has been expressed regarding the opinion that the compressive working stress of reinforced concrete structures should not be more than one-eighth to one-tenth of the ultimate resistance, although no results of experience whatever are cited in support of such an abnormal proposal.

Many tests of concrete in compression, both in America and abroad, show that mixtures as rich as 1 : 2 : 4 may reasonably be expected to give an ultimate compression of at least 2 500 lb. per sq. in. at the end of 3 months, and an increasing ultimate, during the next 1 or 2 years, as great as 4 000 or 4 500 lb. per sq. in., or more. The usual working stresses for such concrete in compression, ranging from 500 to 750 lb. per sq. in. in reinforced concrete work, are thus seen to be conservative. The French Government Committee recommends much higher values.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1076

OVERHEAD CONSTRUCTION FOR HIGH-TENSION ELECTRIC TRACTION OR TRANSMISSION.*

By R. D. Coombs, M. Am. Soc. C. E.

With Discussion by Messrs. Joseph Mayer, W. K. Archbold, Charles Rufus Harte, Farley Osgood, W. S. Murray, and R. D. Coombs.

A proper regard for the safety of the public, and the great necessity, from an operating standpoint, of uninterrupted service on power-transmission lines and the railroad or other lines they cross, demand the most reliable type of construction.

"Crossing spans," being relatively few in number and relatively great in importance, should be built in the best manner possible, and without the same consideration of cost as that which is proper for a line on private right of way.

The choice of a type of construction for transmission companies, on private right of way, may be said to depend on the cost of construction *versus* the cost of interruptions to service, and maintenance. This is also true, though to a less extent, of certain classes of railroad power lines.

Where transmission-company power lines cross railroads, or when high-tension wires are used by railroads for purposes of electric traction, the cost of failures of construction will be found to exceed the additional cost of good construction.

* Presented at the meeting of February 5th, 1908.

Interruptions of service on transmission lines, and accidental contact with other lines, can usually be charged to the following causes:

1st.—Short-circuiting by birds, branches, kites or other objects spanning the wires;

2d.—Burning cross-arms or pins by discharges from the power line;

3d.—Fires burning wooden poles;

4th.—Failures due to lightning;

5th.—Electrical failure of insulators;

6th.—Careless or malicious breaking of insulators;

7th.—Failures due to wind and ice storms, or to floods.

Short-circuiting, while not eliminated, will be reduced as much as appears to be practicable by spacing the wires not less than 30 in. apart and cutting back standing timber from the immediate vicinity of the line.

The burning of pins or poles will be prevented by the use of steel superstructures and metallic pins.

Steel superstructures will not be in danger from lightning, and the addition of a ground wire above the power wires should lessen the likelihood of other lightning troubles on the line.

The electrical failure of insulators is mainly a question of cost.

Malicious injury cannot be guarded against, but may be lessened by a campaign of education.

Mechanical failures, due to storms, may be guarded against in the design; and a properly designed structure may be considered as secure as other forms of steel construction.

Ice Load.—Depending on the climate, allowance must be made for accumulations of sleet upon all wires and superstructures. The latest practice regarding sleet loads varies from no load to a thickness of 1 in. Experience with telephone and telegraph lines indicates that sleet formation ranges from a thin film to a hollow cylinder $2\frac{1}{2}$ in. in diameter and $\frac{1}{2}$ in. thick. It is probable, however, that these large formations are not continuous, or do not remain in place during high winds.

The weight of sleet, particularly the larger formations—which may be partly snow—should be assumed as somewhat less than that of clear ice. It will be noted that a thickness of $\frac{1}{2}$ in. of ice is hereafter

assumed to remain in place during a high wind, the weight being assumed as 0.033 lb. per cu. in.

Wind Loads.—No exact specification for wind pressure has, as yet, been very generally accepted. Perhaps the most common method has been to adopt pressures of from 30 to 50 lb. per sq. ft., as required by bridge specifications, and modify them for cylindrical surfaces. In reality, these wind pressures, as used in bridge practice, include an allowance for vibration, and are not considered as likely to act over extended surfaces.

Since the publication of Sir Isaac Newton's law for the pressures exerted by moving fluids—which, for wind pressures, may be reduced to the form:

$$P = \frac{K}{370} V^2$$

in which P = pressure, in pounds per square foot,
and V = velocity, in miles per hour—

many investigators have experimented, with a view to the determination of values for the constant, K. For normal pressures against thin flat surfaces,* most of the results indicate values between,

$$P = 0.0035\ V^2 \quad \ldots\ldots(1)$$

and $$P = 0.0049\ V^2 \quad \ldots\ldots(2)$$

These formulas, modified to apply to cylindrical surfaces, become

$$P = 0.0021\ V^2 \quad \ldots\ldots(3)$$

and $$P = 0.0029\ V^2 \quad \ldots\ldots(4)$$

The Berlin-Zossen high-speed tests, in which wind pressures against trains were measured, gave the formula,

$$P = 0.0027\ V^2$$

and, using a rounded "nose" on the forward end,

$$P = 0.0025\ V^2$$

In 1903-04, at Niagara Falls, Mr. H. W. Buck† conducted a series of tests on a stranded cable having a span of 950 ft. Dynamometers attached to the center of the cable gave direct readings of the wind pressures occurring in conjunction with velocities indicated by a Government standard anemometer, also placed at the center of the span.

The following limiting conditions are to be noted:

The maximum velocity observed was 40 miles per hour (indicated).

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* See Report of the Special Army Engineer Board, U. S. War Department, Sept. 29, 1894.

† "The Use of Aluminum as an Electrical Conductor," by H. W. Buck, International Electrical Congress, 1904.

The single anemometer used registered the velocities at the center of the span, and gave no indication of the velocities at other points.

It is probable that the stranded cable gave slightly higher results than would be the case with a solid wire.

These tests give the following formula,

$$P = 0.0025\ V^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Omitting from consideration the effects of tornadoes and cyclones, it is necessary to determine, or assume, the maximum velocity of the wind, for general practice or for any particular locality. Many of the results of anemometer tests may be regarded with suspicion, owing to imperfect apparatus, it being very probable that some previously recorded pressures should be reduced from 10 to 25%, to be comparable with those obtained by modern instruments.

Table 1 (U. S. Weather Bureau) shows the equivalent "actual" velocities corresponding to those "indicated" by anemometer readings.

TABLE 1.

Indicated Velocity, in miles per hour.	Actual Velocity, in miles per hour.	Indicated Velocity, in miles per hour.	Actual Velocity, in miles per hour.
0	0.	60	48.0
10	9.6	70	55.2
20	17.8	80	62.2
30	25.7	90	69.2
40	33.3		
50	40.8	*100	*76.2

The records of the United States Weather Bureau—omitting tornadoes, cyclones, and violent gales occurring in some particularly exposed situations—give a maximum indicated velocity of 100 miles per hour. The records at Bidston Observatory (Liverpool, England), from 1884 to 1888, give, as a maximum of ten severe storms, an actual velocity of 78 miles per hour.

Table 2 shows the maximum velocities observed at a number of stations by the United States Weather Bureau:

* Added by comparison.

TABLE 2.

Observatory.	Period.	Maximum Velocity Indicated.	Observatory.	Period.	Maximum Velocity Indicated.
Chicago, Ill.	1871–1906	90	Savannah, Ga.	1894–1903	76
Buffalo, N. Y.	1871–1907	90	Philadelphia, Pa.	1872–1907	75
Galveston, Tex.	1894–1903	84	Bismarck, N. Dak.	1894–1903	72
New York, N. Y.	1871–1907	80	Boston, Mass.	1873–1907	72
Eastport, Me.	1873–1907	78	Salt Lake City, Utah.	1894–1903	60

Table 3 shows the three highest indicated velocities recorded by the United States Weather Bureau at the New York City station, in each year, from 1884 to 1906. The station was moved in March, 1895, from the Manhattan Life Building to the present location at 100 Broadway; the latter is evidently a more exposed position, as shown by the abrupt rise in velocities after 1895. The maximum velocity of 80 miles per hour occurred during a sleet storm.

TABLE 3.

Year.	Date.	Maximum Velocity.	Date.	Maximum Velocity.	Date.	Maximum Velocity.
1884	Oct. 18	44	Feb. 20	40	Dec. 9	40
5	Jan. 17	50	Dec. 7	50	Mar. 10	48
6	Feb. 26	64	Mar. 2	54	Jan. 9	44
7	Dec. 29	50	Nov. 16	48	Feb. 12	46
8	Jan. 26	60	Mar. 5	52	Mar. 13	50
9	Jan. 17	50	Feb. 1	48	Dec. 26	48
1890	Jan. 22	55	Dec. 17	48	Feb. 5	45
1	Dec. 30	53	Mar. 14	45	Jan. 11	44
2	Jan. 26	49	Mar. 11	40	Jan. 5	40
3	Aug. 29	54	Jan. 1	48	Oct. 13	48
4	Apr. 11	48	Oct. 10	48	Jan. 12	43
5	Dec. 27	73	Mar. 28	64	Aug. 4	62
6	Mar. 4	72	Feb. 7	65	Sep. 30	56
7	Jan. 18	60	Feb. 6	60	Oct. 17	60
8	Dec. 4	78	Sep. 7	72	Nov. 11	65
9	Mar. 20	80	Jan. 25	66	Feb. 27	64
1900	Oct. 16	76	Nov. 21	76	Jan. 26	76
1	Nov. 26	72	Jan. 19	72	Feb. 5	70
2	Mar. 19	74	Jan. 1	74	Feb. 2	74
3	July 2	72	Feb. 5	72	Sep. 17	65
4	Apr. 16	73	Sep. 15	68	Mar. 3	65
5	Dec. 10	64	Feb. 7	61	Apr. 10	56
6	Mar. 10	64	Jan. 6	61	Feb. 28	59

Table 4 is a record, by months, of the number of different 12-hour periods during which a maximum velocity of 60 miles, or more, was observed at the New York City station, from 1895 to 1906, inclusive. Inasmuch as a maximum occurring late in one period and another early in the following period are both entered, a few of the entries represent the effects of the same storm.

Tables 3 and 4 indicate that, for the vicinity of New York City:

The maximum velocities occur during the winter months, when sleet may be on the wires.

Indicated velocities of more than 80 miles per hour will rarely, if ever, occur during the life of a given structure.

Indicated velocities of from 65 to 75 miles per hour may be expected several times each year, though much less frequently in conjunction with sleet.

TABLE 4.

Month.	Indicated Velocities, in miles per hour:																Totals.
	60	61	62	63	64	65	66	67	68	70	72	73	74	76	78	80	
Jan.	3	3	....	2	2	1	4	....	....	....	1	....	1	1	....	....	18
Feb.	7	2	2	1	6	2	2	2	2	3	1	....	1	....	....	....	31
Mar.	3	....	1	2	2	1	1	....	1	1	3	....	1	....	....	1	17
Apr.	....	1	....	1	....	....	....	....	....	....	....	1	....	....	....	....	3
May	1	....	1	....	1	....	....	....	....	....	....	....	....	....	....	....	3
June	1	....	....	....	....	....	1	....	....	....	1	....	....	....	....	....	3
July	....	....	....	....	....	2	1	....	....	....	1	....	....	....	....	....	4
Aug	....	....	1	....	1	....	....	....	....	....	....	....	....	1	....	....	3
Sept.	....	....	....	1	1	2	....	....	1	....	1	....	....	....	....	....	6
Oct.	1	1	....	....	....	....	....	....	....	....	....	....	....	1	....	....	3
Nov.	4	....	2	....	2	1	....	....	....	....	1	1	1	1	....	....	13
Dec.	6	1	1	2	3	1	....	....	1	....	1	1	....	....	1	....	18
Totals.	26	8	8	9	18	10	9	2	5	4	10	3	4	4	1	1	122

Assuming an indicated velocity of 100 miles per hour, or an actual velocity of 76.2 miles per hour, Equations 3 and 4 become:

$$P = 12.19 \text{ lb. per sq. ft. of projected area} \quad (6)$$

and $$P = 16.84 \text{ lb. per sq. ft. of projected area} \quad (7)$$

while Equation 5 becomes:

$$P = 14.51 \text{ lb. per sq. ft. of projected area} \quad (8)$$

Equation 8 is a mean of Equations 6 and 7.

On long spans, the maximum pressure at one point may be considerably in excess of the equivalent uniform pressure along the wire, while very short spans may be exposed to the maximum pressure throughout their length. In the absence of a formula in which the length of span enters as a factor, Equation 5 may be regarded as approximately correct.

In view of the rare, if not improbable, occurrence of velocities greater than 90 miles per hour, "indicated," and the further improbability of such winds accompanying sleet storms, or of the sleet remaining in place, Equations 9 and 10 seem to be reasonable for general use.

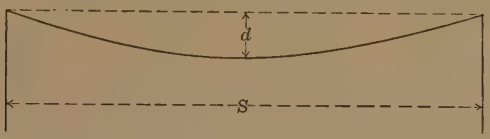
P = 12 lb. per sq. ft. of projected area of bare wires..... (9)

P = 8 lb. per sq. ft. of projected area + ½ in. thickness of ice..(10)

Catenary Stresses.

The following mathematical treatment is not new, but the writer has found the arrangement convenient:

Ends of Span at Same Elevation.



S = span, in feet,

d = sag, in feet,

W = load per linear foot in plane of wire,

A = area of wire, in square inches.

E = modulus of elasticity,

θ = coefficient of expansion,

t = change of temperature, in degrees,

e = elongation or change of length, within elastic limit.

L_o = length, in feet, of imaginary wire ($W = 0$) at normal temperature.

L_{oc} = length, in feet of imaginary wire, cold (80° fahr. below normal temperature).

L_{oh} = length, in feet, of imaginary wire, hot (70° fahr. above normal temperature).

Index to subscripts.—

No subscript = normal conditions,

$_c$ = cold: 80° fahr. below normal + dead load,

$_i$ = cold: ice load + dead load,

$_{cw}$ = cold: wind load + dead load,

$_{iw}$ = cold: ice + wind + dead load,

$_h$ = hot: 70° fahr. above normal + dead load.

W_{iw} is the resultant of the vertical dead + ice loads and the horizontal wind load.

W_{cw} is the resultant of the vertical dead load and the horizontal wind load.

Stresses.—Substitute normal values in Equations 1, 2, 3, and 4. Assume values of T_h, T_{iw}, T_c, T_i, or T_{cw}, such that Equations 5 and 6

will give identical values of d_h, d_{iw}, etc. The tension that will give the same sag by Equations 5 and 6 (independently) is the tension resulting from that sag and the given loading.

$$T = \frac{W S^2}{8 d} \quad \ldots\ldots (1)$$

$$L = S\left[1 + \frac{8 d^2}{3 S^2}\right] \quad \ldots\ldots (2)$$

$$e = \frac{T L}{E A} \quad \ldots\ldots (3)$$

$$L_o = L - e \quad \ldots\ldots (4)$$

(70° fahr. above normal, with dead load.)

$$L_{oh} = L_o (1 + \theta t_h) \qquad e_h = \frac{L_{oh} \times T_h}{E A} \qquad L_h = L_{oh} + e_h$$

$$d_h = 0.612 \sqrt{S (L_h - S)} \quad \ldots\ldots (5)$$

$$d_h = \frac{W_h \times S^2}{8 T_h} \quad \ldots\ldots (6)$$

(80° fahr. below normal, with dead + ice + wind loads.)

$$L_{oc} = L_o (1 - \theta t_c) \qquad e_{iw} = \frac{L_{oc} \times T_{iw}}{E A} \qquad L_{iw} = L_{oc} + e_{iw}$$

$$d_{iw} = 0.612 \sqrt{S (L_{iw} - S)} \quad \ldots\ldots (5)$$

$$d_{iw} = \frac{W_{iw} \times S^2}{8 T_{iw}} \quad \ldots\ldots (6)$$

(80° fahr. below normal, with dead load.)

$$L_{oc} = L_o (1 - \theta t_c) \qquad e_c = \frac{L_{oc} \times T_c}{E A} \qquad L_c = L_{oc} + e_c$$

$$d_c = 0.612 \sqrt{S (L_c - S)} \quad \ldots\ldots (5)$$

$$d_c = \frac{W_c \times S^2}{8 T_c} \quad \ldots\ldots (6)$$

(80° fahr. below normal, with dead + ice loads.)

$$L_{oc} = L_o (1 - \theta t_c) \qquad e_i = \frac{L_{oc} \times T_i}{E A} \qquad L_i = L_{oc} + e_i$$

$$d_i = 0.612 \sqrt{S (L_i - S)} \quad \ldots\ldots (5)$$

$$d_i = \frac{W_i \times S^2}{8 T_i} \quad \ldots\ldots (6)$$

(80° fahr. below normal, with dead + wind loads.)

$$L_{oc} = L_o (1 - \theta t_c) \qquad e_{cw} = \frac{L_{oc} \times T_{cw}}{E A} \qquad L_{cw} = L_{oc} + e_{cw}$$

$$d_{cw} = 0.612 \sqrt{S (L_{cw} - S)} \quad \ldots\ldots (5)$$

$$d_{cw} = \frac{W_{cw} \times S^2}{8 T_{cw}} \quad \ldots\ldots (6)$$

TABLE 5.—Stranded Wire—Steel (Galvanized).

Diameter.	No. and Gauge of Wires.	Area, in square inches.	Ultimate Strength.		
			Siemens-Martin, 75 000 lb.	High-tension, 125 000 lb.	Extra high tension, 187 000 lb.
$\frac{5}{8}$ in.	7— 5	0.2356	19 000	25 000	42 000
$\frac{9}{16}$ "	7— $6\frac{1}{2}$	0.1922	14 500	21 100	34 500
$\frac{1}{2}$ "	7— 8	0.1443	11 000	18 000	27 000
$\frac{7}{16}$ "	7— 9	0.1204	9 000	15 000	22 500
$\frac{3}{8}$ "	7—11	0.0832	6 800	10 500	17 250
$\frac{5}{16}$ "	7—12	0.0606	4 860	8 100	12 100
$\frac{9}{32}$ "	7—$12\frac{3}{4}$	0.0496	4 380	7 300	10 900
$\frac{1}{4}$ "	7—$13\frac{3}{4}$	0.0379	3 050	5 100	7 600
$\frac{7}{32}$ "	7—15	0.0293	2 500	4 100	6 100
$\frac{1}{16}$ "	7—16	0.0218	2 000	3 300	4 900
$\frac{5}{32}$ "	7—$17\frac{1}{4}$	0.0149	1 350	2 230	3 300
$\frac{1}{8}$ "	7—19	0.0097	900	1 500	2 250

Diameter.	Load per Linear Foot (Vertical).		Pressure per Linear Foot (Horizontal).		Load per Linear Foot (Plane of Resultant).	
	Dead.	Dead + ½ in. of ice.	At 8 lb. per sq. ft.	At 15 lb. per sq. ft.	Wind, at 8 lb.	Wind, at 15 lb.
			½ in. of ice.	½ in. of ice.	½ in. of ice.	½ in. of ice.
$\frac{7}{8}$ in.	.....	.....	1.250	2.344	.....	.....
$\frac{13}{16}$ "	.....	.....	1.208	2.266	.....	.....
$\frac{3}{4}$ "	.....	.....	1.167	2.188	.....	.....
$\frac{11}{16}$ "	.....	.....	1.125	2.109	.....	.....
$\frac{5}{8}$ "	.....	.....	1.083	2.031	.....	.....
$\frac{9}{16}$ "	.....	.....	1.042	1.953	.....	.....
$\frac{1}{2}$ "	0.510	1.132	1.000	1.875	1.510	2.190
$\frac{7}{16}$ "	0.415	0.998	0.958	1.797	1.383	2.055
$\frac{3}{8}$ "	0.295	0.839	0.917	1.719	1.243	1.913
$\frac{5}{16}$ "	0.210	0.715	0.875	1.641	1.130	1.790
$\frac{9}{32}$ "	0.144	.....	0.854	1.602	.....	.....
$\frac{1}{4}$ "	0.125	0.592	0.833	1.563	1.022	1.671
$\frac{7}{32}$ "	0.095	0.542	0.812	1.523	0.976	1.617
$\frac{3}{16}$ "	0.075	0.503	0.792	1.484	0.938	1.567
$\frac{5}{32}$ "	0.055	0.463	0.771	1.445	0.899	1.517
$\frac{1}{8}$ "	0.032	0.421	0.750	1.406	0.860	1.468

TABLE 6.—Solid Wire—Copper (Hard-Drawn).

Gauge, B. & S.	Diameter, in inches.	Area, in square inches.	Area, in circular mils.	Ultimate strength, in pounds.	Factor of safety = 2½.
0 000	0.4600	0.1662	211 600	8 310	3 325
000	0.4096	0.1318	167 800	6 590	2 635
00	0.3648	0.1045	133 080	5 220	2 090
0	0.3249	0.0829	105 530	4 560	1 825
1	0.2893	0.0657	83 690	3 740	1 495
2	0.2576	0.0521	66 370	3 120	1 250
3	0.2294	0.0413	52 630	2 480	990
4	0.2043	0.0328	41 740	1 960	785
5	0.1819	0.0260	33 100	1 560	625
6	0.1620	0.0206	26 250	1 240	495
7	0.1443	0.0163	20 820	980	390
8	0.1285	0.0130	16 510	780	310
9	0.1144	0.0103	13 090	620	250
10	0.1019	0.0082	10 380	490	195
11	0.0907	0.0065	8 230	390	155
12	0.0808	0.0051	6 530	305	120
13	0.0720	0.0041	5 180	245	100
14	0.0641	0.0031	4 110	185	75

Gauge, B. & S.	Load per Linear Foot (Vertical).		Pressure per Linear Foot (Horizontal).		Load per Linear Foot (Plane of Resultant).	
	Dead.	Dead + ½ in. of ice.	At 8 lb. per sq. ft.	At 15 lb. per sq. ft.	Wind, at 8 lb.	Wind, at 15 lb.
			½ in. of ice.	½ in. of ice.	½ in. of ice.	½ in. of ice.
0 000	0.641	1.238	0.973	1.824	1.575	2.205
000	0.509	1.074	0.940	1.762	1.427	2.064
00	0.403	0.940	0.910	1.706	1.309	1.948
0	0.320	0.833	0.883	1.656	1.214	1.854
1	0.253	0.744	0.860	1.612	1.137	1.775
2	0.202	0.673	0.838	1.572	1.075	1.710
3	0.159	0.613	0.820	1.537	1.024	1.655
4	0.126	0.564	0.803	1.505	0.981	1.607
5	0.100	0.524	0.788	1.477	0.946	1.567
6	0.079	0.491	0.775	1.453	0.917	1.534
7	0.063	0.464	0.763	1.430	0.893	1.503
8	0.050	0.441	0.752	1.411	0.872	1.479
9	0.039	0.421	0.743	1.393	0.854	1.456
10	0.032	0.406	0.735	1.377	0.840	1.436
11	0.025	0.392	0.727	1.363	0.826	1.418
12	0.020	0.381	0.721	1.351	0.815	1.404
13	0.016	0.372	0.715	1.340	0.806	1.391
14	0.012	0.363	0.709	1.330	0.796	1.379

Ultimate strength = 50 000 to 60 000 lb. per sq. in.
Modulus of elasticity = E = 16 000 000
Coefficient of expansion = θ = 0.0000096

TABLE 7.—Stranded Wire—Copper (Hard-Drawn).

Gauge, B. & S.	Diameter, in inches.	Area, in square inches.	Area, in circular mils.	Ultimate strength, in pounds.	Factor of safety = 2½.
	0.999	0.5892	750 000	.....	.....
	0.963	0.5495	700 000	.....	.....
	0.927	0.5102	650 000	.....	.....
	0.891	0.4715	600 000	.....	.....
	0.855	0.4318	550 000	.....	.....
	0.819	0.3924	500 000	.....	.....
	0.770	0.3523	450 000	.....	.....
	0.728	0.3141	400 000	.....	.....
	0.679	0.2750	350 000	.....	.....
	0.630	0.2360	300 000	.....	.....
	0.590	0.1965	250 000	.....	.....
0 000	0.530	0.1662	211 600	9 970	3 990
000	0.470	0.1318	167 800	7 910	3 160
00	0.420	0.1045	133 080	6 270	2 510
0	0.375	0.0829	105 530	4 970	1 990
1	0.330	0.0657	83 690	3 940	1 580
2	0.291	0.0521	66 370	3 130	1 250
3	0.261	0.0413	52 630	2 480	990
4	0.231	0.0328	41 740	1 970	790

Circular mils, or B. & S. gauge.	Load per Linear Foot (Vertical).		Pressure per Linear Foot (Horizontal).		Load per Linear Foot (Plane of Resultant).	
	Dead.	Dead + ½ in. of ice.	At 8 lb. per sq. ft. ½ in. of ice.	At 15 lb. per sq. ft. ½ in. of ice.	Wind, at 8 lb. ½ in. of ice.	Wind, at 15 lb. ½ in. of ice.
750 000	2.288	3.220	1.338	2.499	.....	.....
700 000	2.135	3.045	1.309	2.454	.....	.....
650 000	1.983	2.871	1.285	2.409	.....	.....
600 000	1.830	2.695	1.261	2.364	.....	.....
550 000	1.678	2.521	1.237	2.319	.....	.....
500 000	1.525	2.345	1.213	2.274	.....	.....
450 000	1.373	2.163	1.180	2.213	.....	.....
400 000	1.220	1.984	1.152	2.160	.....	.....
350 000	1.068	1.801	1.119	2.099	.....	.....
300 000	0.915	1.618	1.087	2.038	.....	.....
250 000	0.762	1.440	1.060	1.988	.....	.....
0 000	0.645	1.286	1.020	1.913	1.641	2.305
000	0.513	1.116	0.980	1.838	1.485	2.150
00	0.406	0.978	0.947	1.775	1.361	2.026
0	0.322	0.866	0.917	1.719	1.261	1.925
1	0.255	0.771	0.887	1.663	1.175	1.833
2	0.203	0.695	0.861	1.614	1.107	1.757
3	0.160	0.633	0.841	1.576	1.053	1.698
4	0.127	0.582	0.821	1.539	1.006	1.645

Ultimate strength = 60 000 lb. per sq. in.

TABLE 8.—Stranded Wire—Aluminum.

Gauge, B. & S.	Diameter, in inches.	Area, in square inches.	Area, in circular mils.	Ultimate strength, in pounds.	Elastic limit.	Factor of safety = 2½.		
	1.035	.....	800 000	.....	60%	.....	23 000 lb. per sq. in.	37 wires.
	0.996	0.5892	750 000	13 550	.....	5 420		
	0.963	0.5495	700 000	12 640	.....	5 050		
	0.928	0.5102	650 000	11 730	.....	4 690		
	0.891	0.4715	600 000	10 840	.....	4 340		
	0.854	0.4318	550 000	9 930	.....	3 970		
	0.814	0.3924	500 000	9 025	.....	3 610		
	0.772	0.3523	450 000	8 100	.....	3 240	"	19 wires.
	0.725	0.3141	400 000	7 225	.....	2 890		
	0.679	0.2750	350 000	6 325	.....	2 530		
	0.621	0.2360	300 000	5 430	.....	2 170		
	0.567	0.1965	250 000	4 520	.....	1 810	" "	7 wires.
0 000	0.522	0.1662	.....	3 820	.....	1 530		
000	0.464	0.1318	.....	3 160	65%	1 265	24 000 lb. per sq. in.	
00	0.414	0.1045	.....	2 510	.....	1 000		
0	0.368	0.0829	.....	1 990	.....	795		
1	0.328	0.0657	.....	1 575	.....	630		
2	0.291	0.0521	.....	1 250	.....	500		
3	0.261	0.0413	.....	990	.....	395		
4	0.231	0.0328	.....	790	.....	315		

Circular mils, or B. & S. gauge.	Load per Linear Foot (Vertical).		Pressure per Linear Foot (Horizontal).		Load per Linear Foot (Plane of Resultant).	
	Dead.	Dead + ½ in. of ice.	At 8 lb. per sq. ft. ½ in. of ice.	At 15 lb. per sq. ft. ½ in. of ice.	Wind, at 8 lb. ½ in. of ice.	Wind, at 15 lb. ½ in. of ice.
800 000	0.736	1.691	1.357	2.544	.....	.....
750 000	0.690	1.620	1.331	2.495	.....	.....
700 000	0.644	1.554	1.309	2.454	.....	.....
650 000	0.598	1.486	1.285	2.410	.....	.....
600 000	0.552	1.417	1.261	2.364	.....	.....
550 000	0.506	1.348	1.236	2.318	.....	.....
500 000	0.460	1.278	1.209	2.268	.....	.....
450 000	0.414	1.205	1.181	2.215	.....	.....
400 000	0.368	1.130	1.150	2.156	.....	.....
350 000	0.322	1.055	1.119	2.099	.....	.....
300 000	0.276	0.973	1.081	2.026	.....	.....
250 000	0.230	0.894	1.045	1.959	.....	.....
0 000	0.195	0.831	1.015	1.902	1.312	2.075
000	0.155	0.755	0.976	1.830	1.234	1.979
00	0.122	0.691	0.943	1.768	1.168	1.898
0	0.097	0.637	0.912	1.710	1.112	1.825
1	0.077	0.592	0.885	1.660	1.065	1.762
2	0.061	0.553	0.861	1.614	1.023	1.706
3	0.049	0.522	0.841	1.576	0.990	1.660
4	0.039	0.494	0.821	1.539	0.958	1.616

Ultimate strength = 23 000 to 24 000 lb. per sq. in.
Modulus of elasticity = E = 9 000 000.
Coefficient of expansion = θ = 0.0000128.

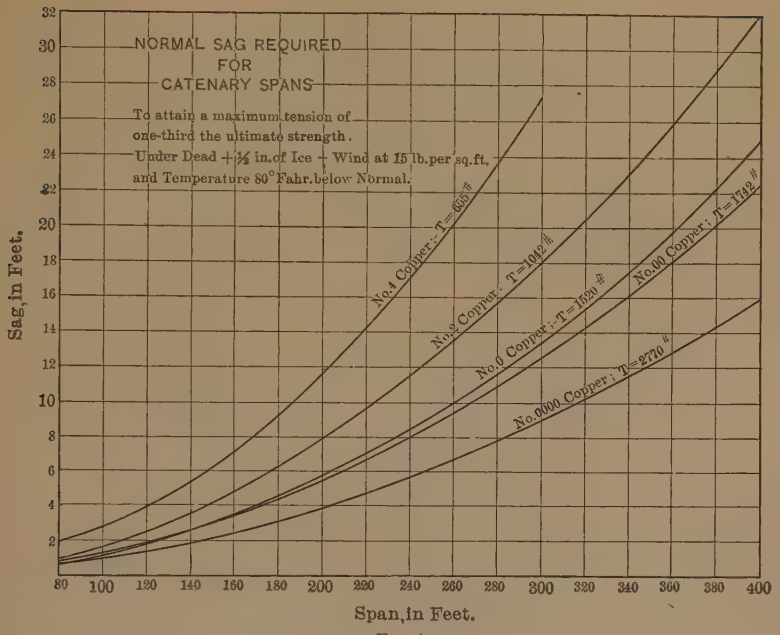
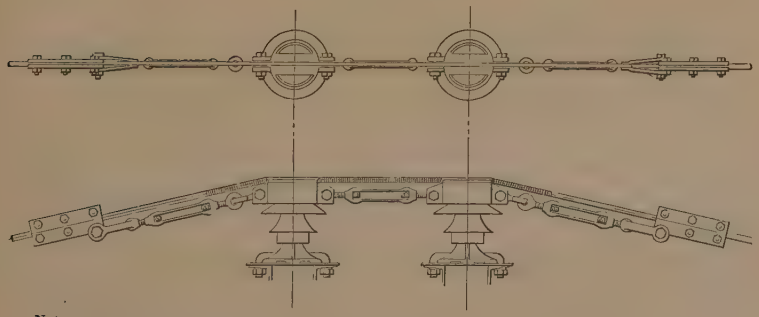


FIG. 1.

ANCHOR INSULATOR



Note:

Cap to be lined up properly and cemented to Insulator.
Neither the Cap nor Cap-cement to rest on Petticoat.
All parts to be galvanized, except inside of Cable-clamps.
Soft Copper Bushing around the wire in the Clamp.
Soft Copper Shield around the wire over the Insulator.
Serving wire No. 12 B. and S. Copper.
All edges and corners to be rounded, particularly at ends of Clamp and Cap grooves

FIG. 2.

Fig. 1 shows a set of curves giving the approximate normal sags required for hard-drawn, solid, copper wire, when the maximum tension is limited to one-third the ultimate strength.

Figs. 2 and 3 show two types of anchor insulators, in which two or more insulators withstand the tension of the span, and in which an auxiliary attachment is provided in case of failure at the insulator.

ANCHOR INSULATOR

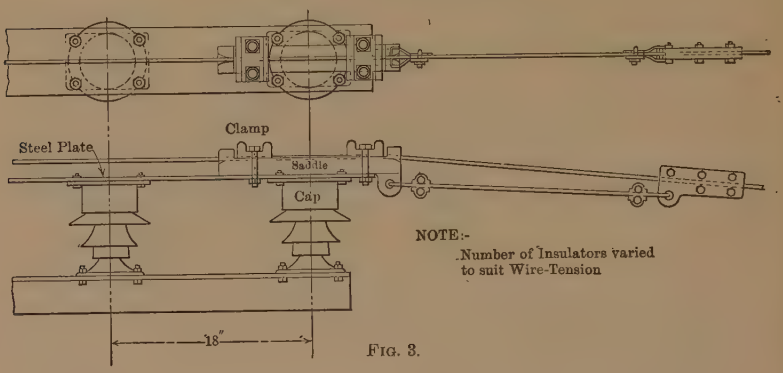


FIG. 3.

SPECIFICATIONS FOR OVERHEAD CONSTRUCTION OF HIGH-TENSION TRANSMISSION-LINE CROSSINGS.

General Requirements.

Drawings.—Complete drawings shall be furnished, in duplicate, for approval before construction is commenced.

General drawings shall contain full information covering stresses, span, normal sag, size and material of wires or cables, voltage, elevation of the points of support from the top of the rail, and the maximum sag and consequent clearance above the top of the rail.

Detailed plans, covering insulators, pins, clamps, etc., and their supporting construction and foundations, shall be furnished for approval.

After approval, (.....) complete sets of drawings shall be furnished for file, and, when the construction is at the expense of the Railroad Company, the original tracings shall be forwarded for file.

Clearance.—The clear distance of any part of the construction from the center line of the track, and the clear head-room above the top of the rail, shall be as specified by the proper official of the Railroad Company, but the clear head-room shall not be less than 30 ft. nor less than 6 ft. above any existing wires, such clearance to obtain under the maximum deflection due to loads and temperature.

Minimum spread of wires.....30 in.
Minimum side or top clearance between wires and superstructure.15 in.
Minimum side clearance between insulators and superstructure.. 8 in.

Spans.—The spans immediately adjoining the crossing span shall be slack spans. Poles or towers at the ends of crossing spans and slack spans shall be self-supporting, or be guyed so as to be in effect self-supporting. Double cross-arms (or an approved substitute), insulators, clamps, etc., at the ends of crossing spans and slack spans shall be designed so as to prevent the wires from pulling through in case of failure in an adjoining span or at the support.

Condition of Loading.—The structure shall be designed to withstand, with the allowable factor of safety, the combined effect of the dead, ice and wind loads, and be self-sustaining under the following conditions of loading:

(1) Dead + ice + wind on adjoining spans.
(2) Dead + ice + wind on one span and dead load on adjoining span.

Loads.—

Dead load, or weight of the material;
Ice load: Weight of ice $\frac{1}{2}$ in. thick all around exposed members. Weight of ice, 0.033 lb. per cu. in.
Wind load: Wind pressure on wires or other cylindrical surfaces, 15 lb. per sq. ft. of projected area, such area being taken at the increased figure due to ice $\frac{1}{2}$ in. thick.
Pressure on flat surfaces, 27 lb. per sq. ft.

Factors of Safety.—Factors of safety, under the combination of loads giving the maximum stress, shall be as follows:

Wires and cables	$2\frac{1}{2}$
Insulators, pins, clamps, steel cross-arms and connections	3
Steel superstructure	$2\frac{3}{4}$
Wooden superstructure and cross-arms	5

Temperature.—In the determination of stresses and clearances, and in erection, provision shall be made for a change in temperature of 70° fahr. above and 80° fahr. below a normal temperature of 60° fahr.

Thickness of Material.—All metal in the superstructure shall be not less than $\frac{1}{4}$ in. in thickness.

Radius of Gyration.—The length of any compression member shall not exceed 150 times its least radius of gyration.

Connections.—Connections, generally, shall develop the full strength of the member, and shall be designed to avoid, as far as possible, induced stresses due to eccentricity.

Net Section.—In calculating tensile stresses, allowance shall be made for reduction in area, due to rivet holes (adding $\frac{1}{8}$ in. to the nominal diameter), screw threads, etc.

Rivets.—Rivets shall be machine-driven, wherever practicable. Loose or defective rivets shall be carefully cut out and replaced; if

necessary, to avoid injuring the material, they shall be drilled out. The diameter of the finished rivet hole shall not be more than $\frac{1}{16}$ in. greater than the diameter of the cold rivet.

Bolts.—Bolts shall not be used in place of rivets, except as specified; and when used, the holes shall be reamed and the bolts made to a close fit.

Straightening.—All material, when necessary, shall be carefully straightened at the shop before assembling.

Drainage.—Pockets, such as enclosed column footings, shall have drain holes, and shall be filled with water-proof material (or concrete), or both, as may be required.

Galvanizing.—Messenger and guy wires, insulator pins, clamps, etc., shall be galvanized. Structural-steel poles shall be galvanized in an approved manner, or shall be painted, as provided below.

Painting.—Structural steel shall be thoroughly cleaned at the shops and given one good coat of linseed oil. All surfaces coming in contact in assembling shall be given one coat of approved paint. Parts which will be inaccessible after erection shall receive two shop coats of approved paint. All machined surfaces shall be coated with white lead and tallow. After erection, all steel shall be given two coats of approved paint. Painting shall not be done during rainy weather, or when the surface of the metal is wet. All dirt, cinders, oil blisters, etc., shall be removed before painting.

Weight.—A variation, in section or weight of materials, of more than $2\frac{1}{2}\%$ will be sufficient cause for rejection, except that sheared plates may vary according to the allowances of the manufacturers' standard.

Foundations.—The foundations shall be designed to resist overturning, assuming:

Weight of "earth"..... 90 lb. per cu. ft.
" " "concrete".....140 " " " "
Angle of friction of "earth".....33° with vertical.

Foundations, in general, shall extend above the ground as a protection to the lower part of the structure; otherwise, the depth and spread of foundations shall be governed by the local conditions.

Guys.—Guy wires shall have an efficient anchorage, and be protected, or of extra strength, at the ground level.

Cradles.—Cradles are not to be furnished, but the crossing shall be designed for their possible future installation.

Timber.—All timber shall be of the best quality of the kind and use specified, cut from sound trees, and sawed to size; close-grained and solid, and out of wind; free from defects, such as injurious ring shakes, crooked grain, large, unsound or loose knots, knots in groups, decay, large pitch pockets or other defects which would materially impair its strength.

TABLE 9.—Unit Stresses in Materials, in Pounds Per Square Inch.

Material.	Tension.	Compression.	Shear.	Bearing.	Bending.	Compression with the grain.	Compression across the grain.	Transverse shear.	Longitudinal shear.	Columns.
Soft steel	17 500	$\left\{\begin{matrix}17\,500 \\ -60\,\frac{l}{r}\end{matrix}\right.$	12 000	.....	.....	.....	.....	.....	.....	.....
Medium steel	20 000	$\left\{\begin{matrix}20\,000 \\ -70\,\frac{l}{r}\end{matrix}\right.$	13 000	.....	.....	.....	.....	.....	.....	.....
Shop rivets and pins	.....	.....	12 000	24 000	.....	.....	.....	.....	.....	.....
Field rivets and bolts	.....	.....	9 000	18 000	.....	.....	.....	.....	.....	.....
Pins	.....	.....	.....	.....	24 000	.....	.....	.....	.....	.....
Long-leaf yellow pine	.....	.....	.....	.....	1 200	1 400	300	1 000	150	$\left\{\begin{matrix}1\,200 \\ -18\,\frac{L}{D}\end{matrix}\right.$
Oak	.....	.....	.....	.....	1 200	1 400	400	800	160	$\left\{\begin{matrix}1\,000 \\ -15\,\frac{L}{D}\end{matrix}\right.$
Chestnut	.....	.....	.....	.....	1 000	1 000	180	300	120	$\left\{\begin{matrix}800 \\ -10\,\frac{L}{D}\end{matrix}\right.$
Cedar	.....	.....	.....	.....	950	1 000	150	300	.....	$\left\{\begin{matrix}800 \\ -12\,\frac{L}{D}\end{matrix}\right.$

TABLE 10.—Composition, etc., of Steel and Iron.

Kind of Steel, etc.	Phosphorus, in acid steel, not more than :	Phosphorus, in basic steel, not more than :	Sulphur, not more than :	Ultimate strength, in pounds per square inch.	Elastic limit. Percentage.	Minimum elongation in 8 in. Percentage.	Minimum reduction in area. Percentage.	Bending.
Medium, open-hearth, structural steel...	0.06%	0.04%	0.05%	60 000 to 68 000	50	22	50	180° cold.
Soft, open-hearth, structural steel.....	0.06%	0.04%	0.05%	52 000 to 60 000	50	25	50	cold, 180° flat.
Rivet steel.....	.....	0.04%	0.04%	48 000 to 56 000	55	28	56	quenched, 180°
Open-hearth cast steel, annealed.....	0.08%	0.05%	0.05%	65 000 Minimum.	50	in 2 in. 15	20	cold, 90° diameter = thickness × 3
Cast iron.....	.....	.....	0.10%	See note*	....	.....	....	.....

* Cast iron, unless otherwise specified, shall be tough, gray iron. It shall be free from flaws and excessive shrinkage. Tests on the "Arbitration Bar" ($1\frac{1}{4}$ in., circular, and 15 in. long), with supports 12 in. apart, shall show a deflection of 0.10 in. for a center load of 2 900 lb.

Each wooden pole or tower shall be set in a concrete base.

Poles shall be of the first quality, with a minimum diameter of 7 in. at the top, and within the following limits of wind:

30 to 40 ft. long—not more than 3 in.
40 to 50 " " — " " " 4 "
50 to 60 " " — " " " 5 "
60 ft. and longer— " " " 6 "

Cross-arms, bearing surfaces, and the surfaces of notches, etc., shall be treated with paint or preservative.

Adjustment.—The sag given the catenaries at erection shall be adjusted to the temperature of the wires, that is, it shall be that corresponding to the dead-load tension of the given span at the temperature of the wires. Particular care shall be exercised in adjusting insulators, clamps, etc., to obtain the desired distribution of stresses.

Workmanship.—The workmanship on the various classes of construction involved shall conform to the requirements of first-class practice.

Insulators.—One or more insulator units, as may be required, shall be assembled complete and mechanically tested to destruction. Insulators shall be tested in accordance with special instructions to be given. Insulators or caps shall not be cemented during freezing temperatures.

Reasonable notices of the proposed tests shall be given the ....., and his representative shall be given every facility for witnessing them.

In all cases full test reports shall be forwarded.

Concrete.

The proportions of the materials in the concrete shall be as called for on the drawings, and the proportioning and methods of mixture shall be as required by the Engineer.

*Cement.**—All cement used in the work shall be of an approved brand of Portland cement. The specific gravity of the cement, thoroughly dried at 100° cent., shall not be less than 3.10. It shall leave, by weight, a residue of not more than 8% on the No. 100 and not more than 25% on the No. 200 sieve.

It shall develop initial set in not less than 30 min., but must develop hard set in not less than 1 hour nor more than 10 hours.

Tensile Strength.—The minimum requirements for tensile strength for briquettes 1 in. square in section shall be within the following limits, and shall show no retrogression in strength within the periods specified:

Neat cement:

24 hours in moist air.....150 to 200 lb.
7 days (1 day in moist air, 6 days in water) 450 to 550 "
28 days (1 day in moist air, 27 days in water) .....550 to 650 "

* Joint Committee on Concrete and Reinforced Concrete.

One part cement, three parts sand:

7 days (1 day in moist air, 6 days in water) 150 to 200 lb.
28 days (1 day in moist air, 27 days in water) .....200 to 300 "

Constancy of Volume.—Pats of neat cement about 3 in. in diameter, ½ in. thick at the center and tapering to a thin edge, shall be kept in moist air for a period of 24 hours.

(*a*)—A pat is then kept in air at normal temperature, and observed at intervals for at least 28 days.

(*b*)—Another pat is kept in water maintained as near 70° fahr. as practicable, and observed at intervals for at least 28 days.

(*c*)—A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely-closed vessel, for 5 hours.

These pats, to pass the requirements satisfactorily, shall remain firm and hard, and shall show no sign of distortion, checking, cracking, or disintegrating.

Sulphuric Acid and Magnesia.—The cement shall not contain more than 1.75% of anhydrous sulphuric acid (SO_3), nor more than 4% of magnesia (MgO).

Sand.—All sand shall be hard, clean, coarse, and sharp, and shall not contain more than 1.5% of clay or other foreign matter. If required by the Engineer, it shall be screened.

Stone.—All stone shall be sound, hard, and durable, and free from dirt and foreign matter, and shall pass through a ring 1½ in. in diameter.

Consistency.—The degree of moisture for mortar, grout, or concrete shall be as required by the Engineer; in general, mortar shall be plastic, grout fluid, and concrete of such consistency that it will quake when being deposited.

Mortar, grout, or concrete which has commenced to set shall not be used in the work.

Placing.—Concrete shall be deposited in the work so that there will be no separation of mortar and stone. It shall be laid quickly in layers, and spaded as may be required. Rock surfaces shall be thoroughly cleaned, and earth surfaces shall be compacted in a satisfactory manner, before concrete is deposited against them. Surfaces of concrete against which fresh concrete is to be laid shall be cleaned and slushed over with grout, and shall be provided with a bond if required by the Engineer.

Forms.—Forms shall be of substantial construction, and designed to preserve the concrete in the form required by the drawings. All exposed surfaces of concrete shall be true to form and surface.

General Clauses.

1.—Every facility for the inspection of materials and workmanship shall be furnished by the contractor; he shall furnish proper testing apparatus, and shall prepare and test such specimens as may be required.

2.—All work shall be subject to the inspection and approval of the Railroad Company's Engineer, and his interpretations of the drawings and specifications, and his decisions as to the quantity or quality of the work, shall be final and conclusive.

3.—The contractor shall remove all falsework, timber, or rubbish incident to his operations, and shall leave the site unobstructed and clean.

4.—The contractor shall bear the cost of any suit which may arise, and shall pay all damages which may be awarded in consequence of the use by said contractor of any patented device in the construction of any work under these specifications.

5.—The contractor shall obtain all necessary permits, and shall assume all risk of accidents to men or materials prior to the acceptance of the finished structure.

Overhead Construction for Electric Traction.

The adoption of electric traction, under the operating conditions prevailing on the present steam roads, introduces a demand for a more substantial type of construction than was necessary for the so-called trolley lines.

Short spans with many supporting poles near the tracks are objectionable in appearance, are a menace to train hands, and are likely to be injured by derailments. With four or more tracks, at standard spacing, poles between tracks will rarely be permitted in the United States, though such an arrangement has been used abroad. Similarly, the superstructures must be self-supporting, or be free from guys parallel to the track. Guys at right angles to the track may be permitted in some localities, but may not be allowable in others, on account of a limited right of way.

Assuming, for present purposes, that men must be permitted to stand on the tops of cars, and that overhead clearance must be given for wrecking cranes and at public crossings, the following clearances must be maintained:

Minimum clear headroom from top of rail to trolley wire	22 ft.	0 in.
" " " " " " " " power "	22 "	0 "
Minimum clearance from center line of main track	9 "	6 "
" " " " yard running track	8 "	6 "
" " " " yard standing track	8 "	0 "

Superstructures may be divided into three classes:

(1)—Two posts or bents supporting a transverse span wire;
(2)—Two posts or bents supporting a transverse truss;
(3)—Single post or bents supporting cantilever brackets.

Of these, the first has been used on various interurban roads, while the recent New York, New Haven and Hartford Railroad installation is the most prominent illustration of the second. The third, which is a heavy type of the present trolley line construction, has not been used in the United States, though it is now in use in Germany, Switzerland and Italy, under more favorable operating conditions than generally obtain on steam roads in America.

Comparing Classes 2 and 3, the former has the following advantages: It has greater rigidity and strength, is better adapted to the erection of catenaries, and permits the telegraph and telephone cables to be widely separated from the power wires. On the other hand, Class 3 offers less obstruction along the tracks, with decreased probability of injury from derailment; less interference with stations, platforms, etc., and permits the addition of tracks without alteration of existing construction.

While two-track bridges can be made comparatively shallow, the depth necessary for four tracks becomes great enough to be an obstruction to the view of upright signals, and the foreground of intermediate trusses will obscure the engineer's view at approximately 1 200 ft.

Catenaries, or the supporting spans for the conductor, are of two general classes: "single," having one messenger cable, with or without an intermediate or "secondary" cable; and "double," in which the conductor is hung by triangular hangers from a pair of messengers. The former type is merely the interurban trolley construction on a larger scale, while the advocates of the latter consider greater rigidity desirable. Up to the present time, these two types have not been suf-

ficiently tested by operation under severe conditions to make possible a definite decision in favor of either. Subject to the successful operation of the trolley pole or pantagraph frame, in connection with the more flexible single catenary, the writer believes the single catenary to be the better type. The single catenary presents a better appearance, offers less obstruction to the view of signals, and costs less, both originally and in maintenance. It will be found advisable to avoid

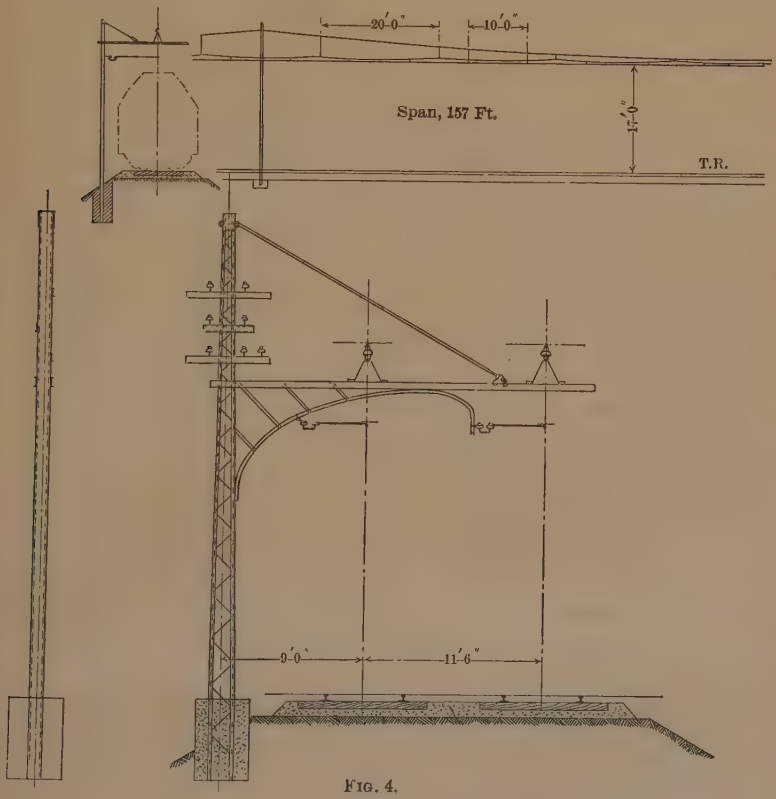


FIG. 4.

many small parts, particularly those of thin section, in catenary construction, inasmuch as, while the need of maintenance should be small, the cost will be relatively high. Work on the ground can be done much more cheaply and easily than that necessitating a work train with telescoping platforms and a right to use the track.

Fig. 4 shows the construction used on several foreign roads.

Catenaries.—The catenary construction is composed of a main messenger cable, from which is hung a secondary messenger, from which in turn the trolley wire is suspended. The main messenger consists of a single catenary, about $\frac{5}{8}$ in. in diameter, of 7-strand, high-tension steel, the normal stress being about 1 100 lb., with an ultimate strength of 11 000 lb., or about 140 000 lb. per sq. in. The secondary messenger is a solid steel wire having a diameter of about 0.236 in., a normal tension of about 220 lb., and an ultimate strength of 6 200 lb., or about 140 000 lb. per sq. in. The trolley wire is grooved, hard-drawn copper, having an area of about 0.155 sq. in. (approximately No. 0000 trolley wire), with an ultimate strength of 8 800 lb., and maintained at a normal tension of about 1 100 lb. by counterweights at intervals of about 1 mile.

The hangers attaching the trolley wire to the secondary messenger are about 6 in. long, and are looped over the messenger, but not fastened to it. There is an allowance of 2 or 3 in. in the loop, so that the trolley wire and hanger can rise vertically, for 2 or 3 in., without raising the messenger. The hanger itself is rigid, and grips the groove in the trolley wire. Counterweights, maintaining constant normal tension in the trolley wire, are spaced at intervals of about 1 mile, at which points the trolley wires lap past each other at an anchor span, each wire passing around pulleys and being attached to counterweights at the post. This arrangement is also used as a combination section-break.

Catenaries are zigzag to the center line of the track, with a displacement of about $\frac{1}{2}$ m. per span between the ends. Care is taken to erect catenaries with the center of the span on the center line of the track, in order to prevent the trolley wire from approaching too close to the ends of the pantagraph bow.

Turnouts are of simple design, having no additional contact wires (gridiron). Turnout catenary and trolley wires are independent spans, and are pulled over into position adjoining the main span. Pull-off posts are used at curves, and the main spans are not reduced on account of curvature. Steady strains are placed at each post, and have an adjustable connection to the short hanger between the secondary messenger and the trolley wire.

The support for the main messenger insulators is of light construction, and is not designed to resist an unbalanced pull. This construction will be made stronger in future work.

The advantage claimed for a catenary of this type, apart from the reduced cost, is extreme flexibility. The counterweighted trolley wire is maintained at constant tension, while being free to move vertically, and therefore presents fewer hard spots against the pressure of the trolley. If the trolley wire tension and the pressure from the bow are carefully adjusted, the wire might be said to hang from the bow and be free to move up and down, within limits, while being supported locally by the passing shoe.

Superstructure.—The superstructure consists of two-track contilever brackets having a guy from the outer end of the bracket to the top of the post. The posts also carry seven power wires, and are composed of two channels with flanges turned inward, with single lacing between the webs of the channels. The cantilever bracket is a horizontal arm with a long curved knee-brace, light channels being used for both members. The post channels are embedded in the concrete foundation. The entire construction is very light, and is not capable of resisting unbalanced loads or broken wires without distortion and without depending on adjoining wires exerting a supporting reaction.

Dimensions.—

Main span	157 ft. 0 in.
Distance of trolley wire from top of rail	17 " 0 "
Distance from center to center of tracks	11 " 6 "
Distance from center of track to center of post	9 " 0 "
Intermediate span of secondary messenger (space between hangers)	20 " 0 "
Distance between trolley wire hangers	10 " 0 "

Operation.—Pressure of bow ..... 12 lb.
Voltage 6 600

Two trolleys are used on each motor car, and, as the overhead construction is very flexible, the bow maintains contact at all times, although it can, and does, deflect the trolley wire vertically. No trouble has been experienced in running past turnouts at a speed of 60 miles per hour, and it is confidently expected that there will be no trouble at higher speeds. The contact shoe lasts for about 4 000 miles.

On the Midland Railway, England, and on other foreign roads, a type of construction similar to the foregoing, but with the following modifications, is about to be installed:

Length of span..... 100 m.
Sag ..... 3 m.
Main messenger—7-strand, high-tension steel; 65 sq. mm.; ultimate strength, 100 kg. per sq. mm. (0.1007 sq. in., or 140 000 lb. per sq. in.).
Secondary messenger—solid steel wire; 6½ mm. in diameter; ultimate strength, 100 kg. per sq. mm. (0.0514 sq. in., or 140 000 lb. per sq. in.).

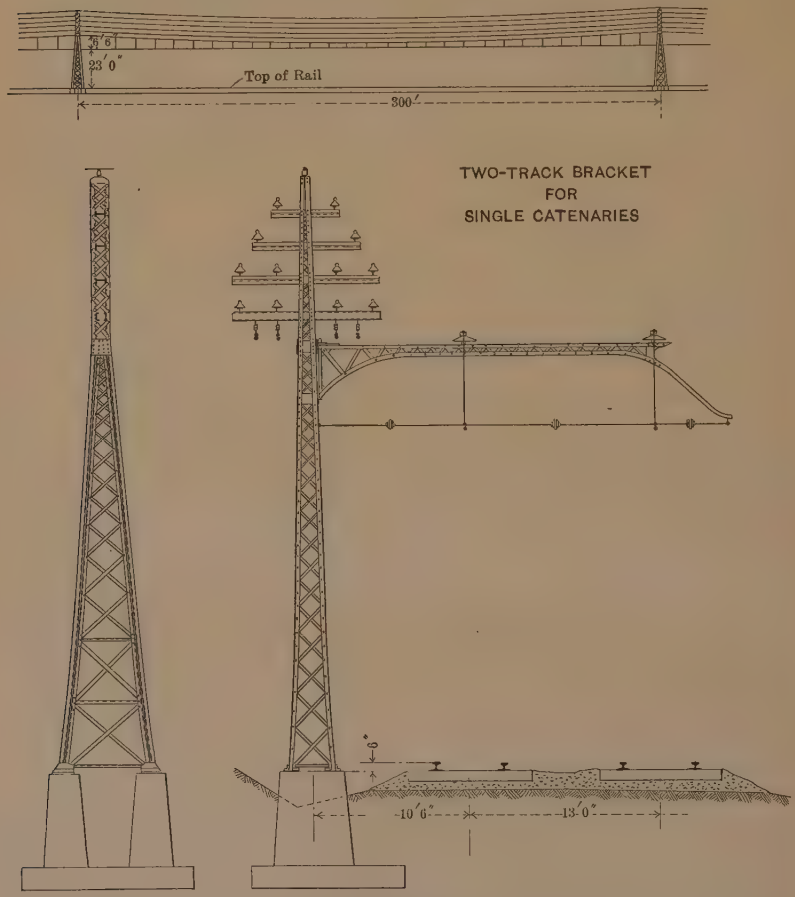


FIG. 5.

Superstructures are to be preferably of the bridge type, instead of cantilever brackets, the overhead "trusses" being light beams (probably channel construction) with supporting guys to the top of the

A-frame posts, and with intermediate posts between the tracks where a number of additional tracks are spanned.

The main catenary insulator supports will be strengthened, and the construction will consist of an insulator supported by the "truss," with

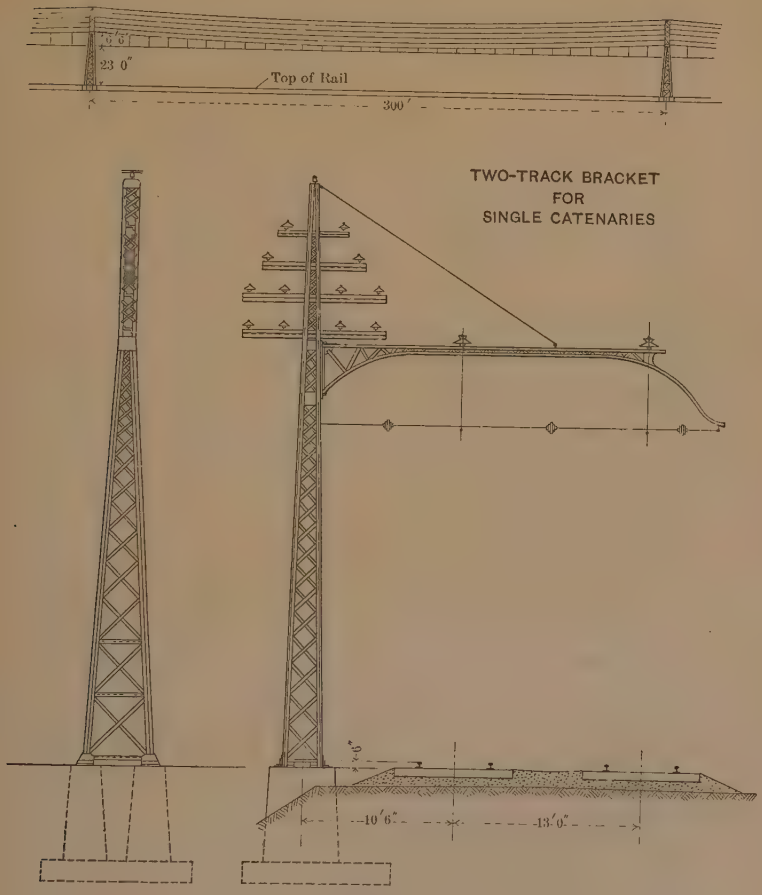


FIG. 6.

a gooseneck carrying another insulator projecting on each side. The main messenger wire will be dead-ended on the insulators at the outer end of the gooseneck.

The writer suggests the types of construction shown in Figs. 5, 6, 7, and 8. As will be noted, these designs are arranged to carry heavy power, telegraph and telephone lines, and may be of two types:

(1).—Cantilever brackets at intermediate points, with anchor and signal bridges spaced about 3 000 ft. apart.

(2).—Bridges for both intermediate and anchor supports.

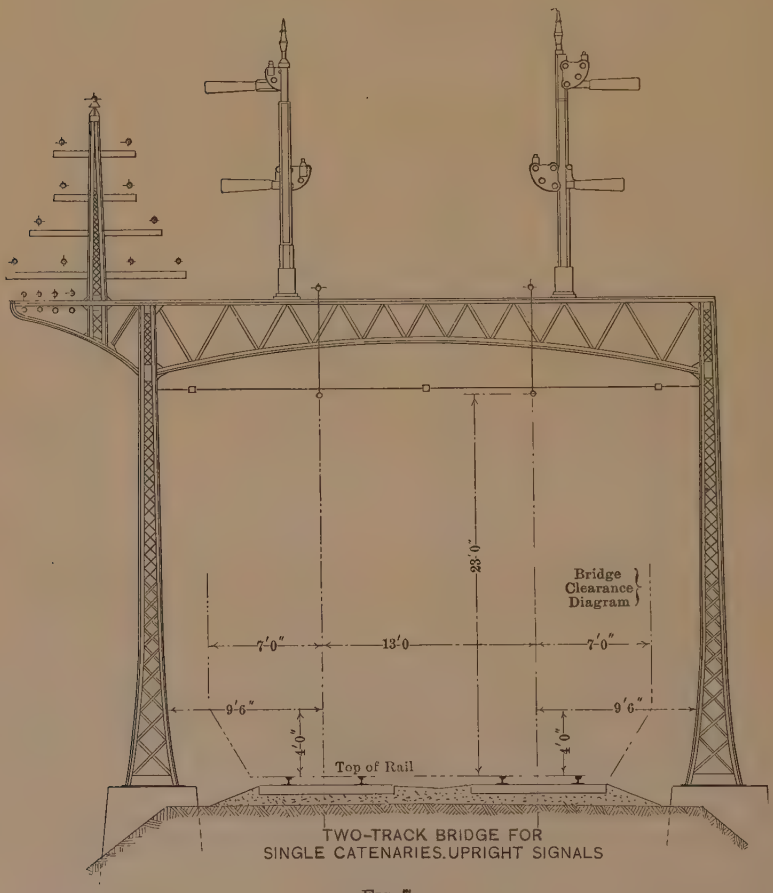


FIG. 7.

General Data.

Spans of 300 ft.,
Single catenaries,
Signal and anchor bridges spaced 3 000 ft. apart,
Power wires (250 000 cm.), stranded copper,
Two 40-pair telephone cables,
Two 25-wire telegraph cables,
No. 0000 trolley wire,
Voltage, 11 000.

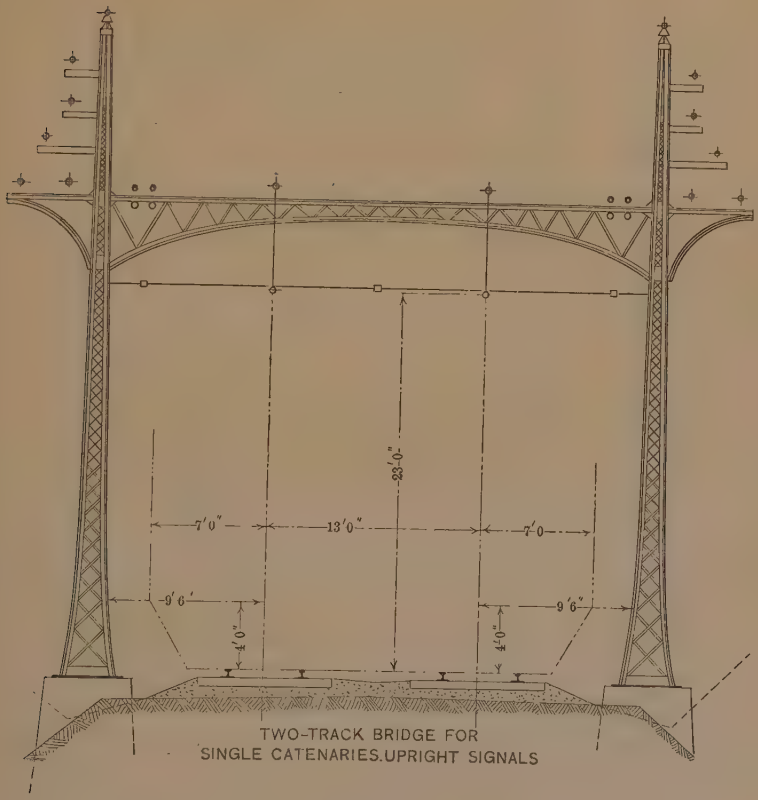


Fig. 8.

Clearances.—

Minimum clearance, top of rail to trolley wire	22 ft. 0 in.
" " " " " " power wires	22 " 0 "
" " " " " " telegraph or telephone cables (except at overhead street bridges, etc.)	22 " 0 "
Minimum spacing, center to center of power wires	2 " 6 "
Minimum uninsulated clearance between wires and superstructure	1 " 3 "
Minimum side clearance between insulators and superstructure	0 " 8 "

Minimum Side Clearance from Center of Track.—

Main-line track	at top of rail	9 ft. 0 in.
	4 ft. or more above rail	9 " 6 "
Yard running track	at top of rail	8 " 0 "
	4 ft. or more above rail	8 " 6 "
Yard standing track	at top of rail	7 " 6 "
	4 ft. or more above rail	8 " 0 "

Ice Load.—Ice ½ in. thick all around exposed members; weight of ice 0.033 lb. per cu. in.

Wind Load.—

8.0 lb. per sq. ft. of projected area of ice-covered wires.
12.0 " " " " of projected area of wires without ice.
20.0 " " " " on flat surfaces.

Conditions of Loading.—

(1).—Dead + ice + 8 lb. wind on adjoining spans.
(2).—Dead + ice + 8 lb. wind on one span; and dead + ice + 2 lb. wind on adjoining spans.
(3).—Dead + 12 lb. wind on one span, and dead + 4 lb. wind on adjoining spans.
(4).—Dead + ice on one span, and dead on adjoining spans.

Factors of Safety.—

Trolley messengers	3
Telegraph and telephone messengers	2
Power wires	$2\frac{1}{2}$
Ground wires	$2\frac{1}{2}$
Steel superstructure	$2\frac{2}{3}$
Insulator pins	3

Ground Wire.—A suitable galvanized, stranded steel wire shall be suspended parallel to and over the group of power wires. The ground wire need not be at the apex of a 60° triangle enclosing all the wires.

Thickness of Material.—The metal in all steel superstructure shall be not less than ¼ in. in thickness.

Temperature.—Provision is to be made in the determination of stresses and clearances, and in erection, for a change in temperature of 70° fahr. above and 80° fahr. below a normal temperature of 60° fahr.

Bolts.—Bolts are not to be used in place of rivets, except as specified, and when used the holes are to be reamed and the bolts made to a close fit.

Galvanizing.—Messenger wires, guy wires, clamps, insulator pins, etc., are to be galvanized in an approved manner.

Unit Stresses.—

Tension	22 500	lb. per sq. in.
Compression	22 500	lb.—82 $\frac{l}{r}$
Shear: Shops rivets and pins	12 000	lb. per sq. in.
Field rivets and bolts	9 000	" " " "
Bearing: Shop rivets and pins	24 000	" " " "
Field rivets and bolts	18 000	" " " "
Bending: Pins	24 000	" " " "
Bearing on concrete foundation	350	" " " "

Design.—If necessary to an economical design, it would be logical to assume higher unit stresses to apply to the case of broken wires, such incidents being presumably of very rare occurrence. It seems to be unreasonable to design the entire line with the normal factor of safety for a condition that will certainly never occur on the whole line, and may never occur on any of it.

It is assumed that the trolley messenger cables, having a factor of safety of 3 under maximum loading, will not break except when burned, and that such burning will not occur under ice and wind load. The result of such an accident would be to rotate the brackets, break some insulator clamps, and probably stop traffic on one 3 000-ft. section, but not to cause failure of the steel superstructure.

The connection of telephone and telegraph cable messengers should be made by swinging hangers, which will allow alterations of adjoining span lengths, and thereby balance unequal tensions in adjoining spans.

Cantilever brackets supporting catenaries should be designed with pivoted or pin-ended connections, in order to permit a horizontal motion of the brackets, and balance any unequal tension in the adjoining catenary spans.

The condition of unequal loading on adjoining spans, due to localized ice or wind loading, does not appear to have been considered by all designers. The conditions of loading given in the specifications, which include a reasonable unbalanced loading, will certainly occur, and should be considered. By a careful arrangement of outline, swinging connections for telephone and telegraph cables, and a small amount of post deflection, the effect of this unbalanced loading on the steelwork may be reduced to approximately the same amount as that of ice and wind on two adjoining spans, which condition of loading is generally accepted.

Superstructures.—The form of the steel supports will depend on local conditions and the line requirements, and may properly be bridges, cantilever brackets, or span wire construction, and may be either built sections or pipe construction.

Foundations.—All foundations should be of concrete.

Trolley Catenaries.—Catenaries, namely, the self-supporting spans of trolley wire, or the supporting spans for a non-self-supporting trolley wire, are of three general classes:

(1).—"Simple catenary," or span of trolley wire;

(2).—"Single catenary," having one supporting messenger cable, with or without an intermediate or secondary messenger;

(3).—"Double catenary," in which the trolley wire is hung by triangular hangers from a pair of messengers.

Simple catenaries, of the type now in use on the various trolley roads, are only available for short spans, because the copper trolley wire cannot withstand the stresses of long spans with small sag. By a modification of the ordinary hanger, Joseph Mayer, M. Am. Soc. C. E., has designed a type of simple catenary which, it is claimed, reduces the stresses in the wire to allowable units.

Simple catenaries and single catenaries, unless provided with lateral guys, are subject to criticism on the score of lateral displacement from wind pressure. Such displacement, in amount sufficient to carry the

trolley wire beyond the range of the contact bow, can only occur locally during severe wind storms. If, however, a locomotive should be in a given span during such displacement, the contact bow would probably rise above the wire and, catching above it, be torn off.

Double catenaries seem to the writer to be less desirable than the foregoing types, for the following reasons:

(1).—Greater first cost of catenaries, insulators, and supporting steelwork;

(2).—More members to maintain, and these of a construction more difficult to maintain;

(3).—Greater mass of material above the tracks, with more unsightly appearance;

(4).—More obstruction to the view of signals, and to the location of signal posts;

(5).—The greater rigidity of the trolley wire does not seem to be a point in favor of double catenaries, but, on the contrary, to cause sparking at the contact bow.

Assuming, then, that either "simple" or "single" catenaries are desirable, it becomes necessary to determine which one of the following types is best suited to meet the conditions of first-class traffic:

(1).—Simple catenary (Mayer's saddle suspension);

(2).—Single catenary;

(3).—Single catenary with secondary messenger (German method).

The writer does not consider that, as yet, a definite decision can be made in favor of any one type, particularly as the result of experience under actual operating conditions is the only satisfactory proof. It is not necessary to obtain some saving in first cost, which must be relatively slight, but it is very important to obtain a construction fulfilling certain requirements.

The maximum stresses in catenary or trolley wire produced by ice and wind loads, temperature changes, or the pressure of the contact bow at high speeds, should not exceed a safe percentage of the elastic limit of the material. There should be sufficient lateral and vertical rigidity to prevent undue displacement from the pressure of the contact bow. The weight, sag, and factor of safety may be chosen so as

to reduce somewhat the likelihood of lateral displacement from wind pressure, otherwise the relative importance of providing lateral rigidity is to be considered.

The trolley wire should have some flexibility, in order to maintain contact with the collector shoe, unless, as seems improbable, a support for the collector shoe can be devised which will adjust itself to all inequalities in the trolley wire. It will be necessary to eliminate, as far as possible, the formation of "hard spots" in the trolley wire, due to the hangers. Local bends may be the cause of sparking, whether they are formed during erection, or by displacement, or by the contact pressure. Simple details, readily replaced, and avoiding the use of thin metal sections, will materially reduce the cost of maintenance.

A successful type of catenary should be adapted to changes in elevation caused by overhead street bridges, without introducing marked differences in the action of the contact bow against the trolley wire.

Pantagraph, or Supporting Frame for the Collector.—There is a very close relationship between the characteristics of a trolley wire and those necessary in a pantagraph frame which will work successfully on that wire.

The use of two pantagraphs, or two collectors, per locomotive will materially reduce trouble from sparking, simultaneous jumping of both contacts being improbable.

The ideal type of pantagraph or supporting frame would be one having rigidity against lateral and longitudinal wind pressure, simplicity of construction, flexibility, little mass or equivalent weight pressing against the trolley wire, and at the same time the ability to maintain contact.

Aside from the sparking at the contact, the possible wear on the trolley wire and on the pantagraph frame, a successful design should keep the wear on the contact shoe within reasonable limits.

DISCUSSION.

Mr. Mayer.

JOSEPH MAYER, M. AM. SOC. C. E.—This paper, in the main, is an interesting collection of data and tables, useful in the design of overhead contact and transmission lines.

The tables of wind velocities and pressures are especially useful for forming a correct opinion of the actual pressures. More stress might be laid on the fact that the observations of the Weather Bureau are made on the tops of high buildings, while the transmission lines, and especially the contact lines, are near the surface of the ground, where the wind pressures are much less. It would also be reasonable to assume less ice on the contact wire than on steel carrying strands, especially on lines of large traffic. Mr. Coombs' recommendations, in regard to wind pressures and unit strains, are generally reasonable. His paper, however, in common with most writings on the same subject, suffers from an insufficient consideration of the bending strains in the wires. In many designs, these bending strains are greater than the tensions, and their neglect leads inevitably to the selection of unsafe designs. They vary so greatly in amount that they cannot be provided for by neglecting them and adopting a large, but uniform factor of safety.

Mr. Coombs suggests for discussion a suspension from a single steel strand with 300-ft. spans and a distance of 6½ ft. from the points of suspension of the strand to the wire. Taking the wire as horizontal, and at the highest temperature, this, with a distance of 6 in. from the lowest point of the strand to the wire, gives a maximum vertical deflection of 6 ft. for the strand. He prefers this to the double catenary and strandless suspension. What justifies this preference, and what makes a superior suspension?

To have adequate conductivity, the wire must be copper; to give a smooth path for the bow, it must be solid; it is impossible to make it straight, but, under the pressure of the sliding bow, especially with high train speeds, it must have a large minimum radius of vertical curvature. The wire should not deviate horizontally far from the center line of track, or, if hung on the side, from a line parallel to it. It must remain safely suspended under the influence of its weight and that of sleet, the wind pressure, the pressure of the sliding bow, and the changes of temperature. The wire and its supporting structure should interfere as little as possible with the view of the signals. These ends should be attained by the simplest means, entailing the least cost of construction and maintenance. The bow lifts the wire, and the curvature of its motion, and not that of the freely hanging wire, must be considered. If the wire is supported at short intervals, it is lifted by the passing bow to positions above its supports; if these

Mr. Mayer. are rigid, the bow, at high train speed, oscillates rapidly up and down. Excessive bending strain in the wire, and jumping of the bow, with sparking, may result.

To ascertain whether the bow will run smoothly, its equivalent weight, the train speed, the cross-section of the contact wire and its tension, the distance apart of the suspenders, their weight, and that of the carrying strand, and the nature of the connection of the wire to the suspenders must be known. For high train speeds, a small equivalent weight of the bow is essential. Long bows are inevitably heavy; high train speeds, therefore, require short bows and small lateral deflections of the contact wire. The equivalent weight of the bow, however, depends even more on the design adopted than on its length. If smooth running is called for, at high speed, with an inferior heavy bow, short spans are inevitable with all suspensions. The rapid vertical oscillation of the bow is avoided in the strandless and the Siemens-Schuckert suspension described by Mr. Coombs.

In the strandless suspension, with long spans, there is a large change in the direction of motion of the bow at the infrequent suspenders; to make this practicable at high speed, the curvature of this motion must be chosen so that where it is convex downward the bow will not jump, and where it is convex upward neither the wire nor the bow nor the suspender will suffer from the increased pressure. In all suspensions there are large changes in the direction of motion of the bow at low overhead crossings and tunnel entrances, and sometimes at grade crossings. With inferior, heavy sliding bows, a perfect design of the contact line for high speeds at these points is difficult or impracticable. A suspender fitted to change the variable direction of approach of the sliding bow into another variable direction of its departure, by a transition curve of large least radius at all temperatures, is here needed with all suspensions.

To obtain a safe wire, its maximum strain must nowhere and never exceed about three-fourths of its elastic limit. It is exposed to bending strains and tensions. The former are often much larger than the latter. In catenary suspensions, a large grooved copper wire, about 0.3 in. wide and 0.6 in. in height of cross-section, is suspended from steel strands made up of very small wires. The copper wire has an ultimate strength of from 50 000 to 60 000 lb., the steel wires, 140 000 lb. or more. The modulus, E, of copper wires is 16 000 000, that of steel strands is 26 000 000 lb. per sq. in. of solid section. It is evident that the bending accompanying changes in vertical and horizontal deflections will produce much more serious bending strains in the large and weak copper wire than in the small and strong steel wires. For calculating them, the vertical and horizontal deflections of the wire under all conditions of load, wind pressure, pressure of the sliding bow, and changes of temperature must be determined. It is

easy to provide steel ropes strong enough to carry the wire, the ice loads, and the wind pressures. The main difficulty arises from their expansion and contraction caused by changes of temperature and tension. These and the wind pressures cause lateral and vertical curvature of the contact wire. Both the deflections and the drop of temperature increase its tension and produce at certain points large bending strains. To determine the degree of safety of the various suspensions, the largest bending strains and tensions in the contact wire and the ropes must be calculated. With regard to obstruction to the view of the signals, the fewer ropes, suspenders, posts, and bridges or brackets, the better. Mr. Mayer.

For judging the suspension suggested by Mr. Coombs by these standards, and for finding whether it is sufficiently rigid to be suitable for use with a sliding bow that will run smoothly at high speed, and sufficiently strong to resist with adequate safety the incident forces, the size of the steel strand and the distance and nature of the suspenders must be reasonably assumed, and the deflections and consequent bending strains and tensions calculated.

Following Mr. Coombs' specification, a $\frac{3}{4}$-in. steel strand is ample, and weighs 0.89 lb. per ft. of span. The 0000 copper wire weighs 0.64 lb., and gas-pipe suspenders, 12 ft. apart, about 0.33 lb., giving a total weight of 1.86 lb. per ft. of span. Ice $\frac{1}{2}$ in. thick on all parts weighs 1.63 lb., giving a total weight, with ice, of 3.49 lb. per ft. of span. Taking the wire to consist approximately of two 0 wires directly above each other, it is 0.65 in. high and half as wide. The wind pressure on the bare metal of wire suspenders and strand, taking 12 lb. per sq. ft. all through, is 1.63 lb., and that on the ice-covered structure, with 8 lb. per sq. ft., 2.57 lb. per ft. of span. Assuming a tension of 1 000 lb. in the contact wire, the lateral deflection of the wire, at maximum temperature, would be 3.15 ft. This assumes that the wire is held at the ends of the spans by steady braces. If these were absent, the lateral deflection would be much larger and the needed sliding bow would be altogether impracticable.

Where sliding bows are used, the wire must run alternately to the right and left of the center line of track so as to distribute the wear over a considerable length of the bow. Taking the lateral displacement of the wire at the brackets to be 1 ft., and allowing $\frac{1}{2}$ ft. for the lateral vibration of the sliding bow, the latter must be 6.96 ft. long to catch the wire, with the strongest winds assumed. A sliding bow of this length, which will not jump at the suspenders, with moderate train speeds, can be designed.

For the highest present steam railway speeds, a much larger tension in the contact wire or more frequent suspenders are needed to prevent jumping and sparking, with the usual connection of the wire to the suspender. Jumping and sparking might also be prevented by

Mr. Mayer. a contrivance allowing the wire to rise at the suspenders, without lifting them, when the sliding bow passes. With steady braces, which are practically unavoidable with this design, the wire carries, at maximum temperature, 0.49 lb. of the total wind pressure of 1.63 lb. per ft., to the steady braces, and its tension is thereby increased. Much more serious is the bending strain in the wire at the clamps which connect it to the steady braces. These clamps may be designed to avoid bending strain in the wire at maximum temperature without wind. In this case, the lateral bending strain in the wire at the end of the clamp, at highest temperature and wind pressure, is 35 300 lb. per sq. in. For the wire here assumed, this bending strain is given by the formula, $s = \frac{19\,200\ Q\ h}{\sqrt{T}}$ where s is the bending strain, in pounds per square inch, and T is the tension in the wire. If T is decomposed into a component having the direction of the wire at the end of the clamp and one normal to it, the horizontal component of the latter is $Q\ h$, and the vertical component $Q\ v$. For the vertical bending of the same wire, the formula is $s = \frac{18\,000\ Q\ v}{\sqrt{T}}$. At the same time, with this bending strain of 35 300 lb. per sq. in., the tension in the wire is 10 800 lb., giving a combined strain of 46 100 lb. per sq. in.

The passing sliding bow increases the total strain to nearly 48 000 lb. per sq. in. The horizontal bending strain may be reduced, theoretically, to one-half, by using two steady braces at each bracket and connecting them to the wire by hinged clamps, the hinges being vertical. The steady braces themselves should have horizontal hinges permitting the clamps to rise and fall with changes of temperature and wind pressure and the passing of the sliding bow. By this rise and fall, the principal vertical bending strains of the wire, except those due to the passing bow, are transferred to the nearest suspender; therefore, they need not be added to the horizontal bending strains occurring at the steady braces. With hinged double steady braces, connected to the wire by hinged clamps, the total strain per square inch at the highest temperature, with the assumed wind pressure, is approximately 30 500 lb. This assumes that all the hinges work without friction. The friction of the hinges may increase this strain considerably. At the lowest temperature, without ice, and with the largest wind pressure, the lateral deflection of the wire is 15 ft., its tension is 5 000 lb., or 30 080 lb. per sq. in.

The bending strain at the steady braces due to horizontal bending is 27 200 lb. per sq. in. with single, and half as much with double, braces and hinged clamps. This gives combined strains of 57 280 and 43 680 lb. per sq. in. These strains are both increased about 1 000 lb. by the passing sliding bow. Since they exceed the elastic limit, the

wire will bend before the strains reach the amount calculated. Repeated forward and backward bending will produce rupture. To reduce these large bending strains and tensions in the contact wire, smaller deformations must be obtained. These can be secured by smaller deflections of the carrying strand which requires either heavier strands or shorter spans. Mr. Mayer.

With a $\frac{7}{8}$-in. strand of 3 ft. maximum vertical deflection and 300 ft. span, carrying 6 in. below the strand at the center of the span a 0000 grooved wire which is horizontal and has 1 000 lb. tension at the highest temperature, without wind, the deflections and strains in the wire are as follows: At the highest temperature and wind pressure, the horizontal deflection of the wire is 2.17 ft., its upward deflection is 0.23 ft., the tension is 8 260 lb. per sq. in., the horizontal bending strain is 20 580 lb. per sq. in. with single and half as much with double steady braces. This gives, with the latter, a combined strain of 18 550 lb., which is increased to about 20 500 lb. per sq. in. by the passing sliding bow. This is a great improvement over the corresponding 30 500 lb. with 6 ft. deflection of the strand.

At the lowest temperature, with the highest wind pressure, without ice, the vertical deflection of the strand is 1.85 ft., its lateral deflection is 0.9 ft. The vertical upward deflection of the wire is 1.18 ft. and its lateral deflection 1.05 ft. The tension in the wire is 4 760 lb., or 28 640 lb. per sq. in.; the horizontal bending strain, with double steady braces, is 9 290 lb., giving a combined strain of 37 930 lb. per sq. in. This is increased to about 39 000 lb. by the passing sliding bow. With hinged steady braces, most of the vertical bending strain, amounting, if concentrated, to 24 800 lb. per sq. in., is transferred to several of the nearby suspenders. If these suspenders have clamps attached to them by horizontal hinges, allowing oscillation of the wire in a vertical plane, then the vertical bending strains are certainly smaller than the horizontal ones; if there are no such hinges, such strains are probably larger than the horizontal bending strains with double steady braces, but they cannot easily be calculated with accuracy. With ice and wind, at the lowest temperature, the lateral deflection of the wire is 1.48 ft., the upward deflection is 0.46 ft., its tension is 4 760 lb. or 28 640 lb. per sq. in. The horizontal bending strain, with double steady braces and hinged clamps, is 13 100 lb., giving a combined strain of 41 740 lb. per sq. in. This is increased about 1 000 lb. by the passing bow.

The maximum tension in the strand is 16 200 lb., which gives a factor of safety of 3, provided the small bending strains are neglected. A sliding bow 5 ft. long is needed, and can be designed so as to give smooth running at all but the highest speeds. It is evident that the strains in the wire are still excessive where the wind pressures and ice loads prescribed by the specification really occur. As these are of

Mr. Mayer. rare occurrence, a structure of this design, with improved hinged double steady braces, connected to the wire by hinged clamps, will probably, in most situations, last a number of years. It would have about the rigidity of a double catenary suspension of the same span with two strands of $\frac{9}{16}$ in. diameter and 6 ft. vertical deflection, having the wire 6 in. below the lowest point of the strands. The largest lateral deflection of the wire of this latter suspension, with the wire tension and wind pressure here assumed, is approximately 2.25 ft. In the double catenary suspension, no steady braces are used, the bending strains in the wire due to its vertical and lateral deflection are distributed to several clamps near the ends of the spans. They cannot easily be calculated with accuracy, but are probably somewhat smaller than in the best single catenary suspension of the same span. The tensions in the wire are nearly the same in both designs here compared. The double catenary suspension is certainly superior in strength to a single catenary suspension of the same span and lateral deflection with single steady braces. All these designs are far inferior in safety to railroad bridges.

Taking now a 0000 round wire, hung from special suspenders, with 300-ft. spans and 4 ft. maximum vertical deflection, with strain adjusters 1 mile apart, the adjusters changing the length of the spans four times a year, so that the variation of temperature with one length of span does not exceed 84° fahr.: The largest lateral deflection of the wire is 2.54 ft. A sliding bow 6 ft. long is required. The tension in the wire at the lowest temperature, with wind and without ice, is 3 260 lb. or 19 600 lb. per sq. in. The maximum bending strain at the same time is 6 250 lb., giving a total strain of 25 850 lb. per sq. in. The corresponding strain in the suspension suggested by Mr. Coombs is 44 680 lb. per sq. in. In the single catenary suspension, with 3 ft. maximum vertical deflection of the strand, it is 39 000 lb. with the best design.

With a coating of ice, $\frac{1}{4}$ in. thick on the contact wire, increasing its diameter $\frac{1}{2}$ in., the maximum tension at the lowest temperature, and with a wind pressure of 8 lb. per sq. ft. of ice-covered wire, is 21 530 lb., the bending strain is 6 250 lb., giving a total of 27 780 lb. per sq. in. A greater thickness of ice on the contact wire would make it difficult to collect the current. Where there is considerable traffic, the wire will be generally several degrees warmer than the atmosphere, and less ice will form on it than on steel strands carrying but little current, and the passing sliding bow will knock off much of that which forms. It is reasonable, therefore, to assume a smaller amount of ice on the contact wire than on the strands. The maximum strain in the wire, with ice $\frac{1}{2}$ in. thick, and a wind pressure of 8 lb. per sq. ft., at the lowest temperature, in the best of the single catenary suspensions of the same span is 42 700 lb. per sq. in.; this would be but

little reduced by assuming the ice on the contact wire ¼ in. thick, the strain without any ice being 39 000 lb. Mr. Mayer.

The strandless suspension here described requires, for smooth running with a speed of 70 miles per hour, a sliding bow of 4 lb. equivalent weight, 6 ft. long. Such a bow can easily be designed, but, as far as the speaker is aware, it is not at present in the American market. The bows in use are designed for smaller speeds. If they are to be used with high speeds, shorter spans are necessary. The 4-ft. deflection of the contact wire requires a larger range of vertical motion of the bow than the catenary suspension of the same span, in which the height of the wire varies only about 2 ft. Though 300-ft. spans are entirely practicable and safe, with improved strandless suspension and a speed of 70 miles per hour, they will not give continuous contact at this speed without improved sliding bows.

The speaker has invented another suspender, which can be used at any speed with ordinary sliding bows of large equivalent weight, and reduces still further the bending strains. With it, 300-ft. spans can be safely used. As the sliding bow may be heavy, it may be made longer, and a maximum vertical deflection of 4½ or 5 ft. may be adopted, thus reducing greatly its maximum tension. This suspender will be described later.

The calculation showing the excessive bending strains in the single catenary suspensions of 300-ft. span is confirmed by practical experience with long-span electric transmission lines. In these, solid wires were first used, but they broke at the insulators even with moderate tension per square inch. Stranded wires, therefore, are now used with long spans. Many experiments have been made with single catenary suspensions. As a result, the present practice in America and in Europe, as far as known to the speaker, does not show any existing spans of more than 160 ft. The importance of reducing the lateral and vertical deflection of the contact wire, and thereby its bending strains, is fully appreciated by the designers of most, if not all, of the existing structures in America. The sliding bows are generally 4 ft. long, and could not be used with large lateral deflections.

Taking a design with 150-ft. spans of a steel strand of $\frac{7}{16}$ in. diameter, with 16 in. maximum vertical deflection, the 0000 grooved wire being horizontal at the highest temperature and 4 in. below the strand at the center of the span, the following deflections and strains are obtained with a variation of temperature of 140° fahr., and the loads and wind pressures mentioned by Mr. Coombs: The weight of wire, strand and suspenders is approximately 1.04 lb. per ft., the wind pressure, without ice, is 1.15 lb. per ft. The weight, with ice, is 2.33 lb., and the wind pressure is 2.10 lb. per ft. With the greatest wind pressure, and at the highest temperature, the lateral deflection of the

Mr. Mayer. wire is approximately 0.95 ft., its tension is 1 284 lb., or 7 730 lb. per sq. in., the horizontal bending strain, with ordinary steady braces, is 17 410 lb., giving a combined strain of 25 140 lb. per sq. in.; this is increased to about 27 100 lb. by the passing of an improved sliding bow of a maximum dynamic pressure of 25 lb.

At the lowest temperature, with the greatest wind pressure, and without ice, the upward deflection of the wire is approximately 0.3 ft., the horizontal deflection is 0.38 ft., its tension is 4 640 lb., or 27 920 lb. per sq. in. The horizontal bending strain is 6 630 lb., giving a combined strain of 34 550 lb. per sq. in. This is increased to about 35 500 lb. by the passing sliding bow. If the wire is firmly held at the steady brace, so that it cannot rise and fall, the combined strain due to tension and horizontal and vertical bending is about 40 000 lb.

With ice having an average thickness of ½ in., at lowest temperature and with the greatest wind pressure, the wire has a lateral deflection of 0.6 ft. and a downward deflection of 0.07 ft. Its tension is 28 160 lb., and the horizontal bending strain is 10 510 lb., giving a combined strain of 38 670 lb. per sq. in. This is increased by about 1 000 lb. per sq. in. by the passing sliding bow. The horizontal bending strain may be reduced to one-half and the vertical bending strain transferred, by a perfect double steady brace, allowing the wire to rise and fall and turn.

Wire having an elastic limit of 40 000 lb. per sq. in. can be obtained, and, with little ice and moderate wind pressures and changes of temperature, gives an approximately safe structure with the usual designs.

The maximum tension in the steel strand, with the foregoing loads, is 5 000 lb.; the ultimate strength of a cast-steel strand is about 13 600 lb.

These results explain why much longer spans are not used with single catenary suspension.

It is evident that a structure having the factor of safety of a railroad bridge is not practicable with ordinary catenary suspension, in most climates. If Mr. Coombs' description of the Siemens-Schuckert suspension is correct, and if the pulleys over which the contact wire is carried have a diameter of 12 ft., larger pulleys being impracticable, the bending strain in the wire here assumed would be 36 000 lb. per sq. in. If the tension in the contact wire is made small, the lateral deflection of the wire would be much increased, and long spans with large strand deflection would be impracticable. This suspension is used with spans of 48 m., with a steel carrying structure where long spans are very desirable. If the designers had thought them practicable, they would probably have adopted them. Spans of 300 ft., with single catenary suspension, therefore, are not sustained by precedent; they cannot be defended successfully by theory, and

they will probably prove short-lived if tried under conditions approximating those here assumed. Mr. Mayer.

The maximum strains which demonstrably exist in the contact wires with catenary suspension show what a copper wire can stand, at least for a few years. They make it extremely probable that a wire in which the maximum strain never and nowhere exceeds 30 000 lb. per sq. in. is abundantly safe.

W. K. Archbold, Esq.—Regarding the matter of protective structures where transmission lines cross railroad tracks, Mr. Coombs' paper should help to standardize the practice, which has varied extremely, as the speaker has had occasion to note. Under the direction of Thomas H. Mather, M. Am. Soc. C. E., an overhead construction has recently been designed and installed on the line of the Syracuse, Lake Shore and Northern Railroad, running from Syracuse to Baldwinsville, N. Y. The line is about 5 miles long, and is provided with single-catenary trolley construction presenting some new features. Mr. Archbold.

Mr. Mather thinks that the work has not yet advanced far enough to warrant the presentation of a formal paper, and therefore the speaker will make simply a preliminary presentation of the prominent features of the construction.

The trolley wire is hung from a messenger cable supported on bridges spaced 300 ft. apart, from center to center. The bridges, as shown in Plate LXIV, consist of light trusses on bents 30 ft. apart, from center to center. The bents are each composed of two 8-in. channels, 6 ft. apart at the base, converging to 8 in. at the top, and supported on concrete pedestals, 20 in. square, and of depth varying with the nature of the ground. The trusses have an 8-in. channel top chord and 6-in. channel bottom chords, set with the flanges down. The diagonal members are $\frac{5}{8}$-in. rods, and the struts $2\frac{1}{2}$ by $2\frac{1}{2}$ by $\frac{1}{4}$-in. angles. The struts are flattened and bent over at the ends, and are riveted to the channels.

To the top chord of each truss are bolted malleable-iron pins to which are cemented porcelain insulators for the messenger cable. The three-phase high-tension line is supported on steel **A**-frames at each end of each bridge. The construction is designed for a wind pressure of 8 lb. per sq. ft. on the trolley and messenger cables, covered with $\frac{1}{2}$ in. of ice, a somewhat lower ice load being assumed on the high-tension cables, which are of No. 2 copper. The structure is computed as a braced portal, the unit strains under the assumed wind and ice load being 22 500 lb. per sq. in., reduced for compression members.

The catenary is strung for a net sag of 6.5 ft. at 100° fahr. At 20° fahr., the sag is about 5.5 ft., and the trolley is about 1 ft. higher at the center of the span than under the bridges, the height from rail to trolley being 18 ft. at the bridges. Stranded steel messenger, 15 000-lb. wire, $\frac{7}{16}$ in. in diameter, supports steel hangers, $\frac{5}{8}$ in. in diameter,

Mr. Archbold. spaced 10 ft. apart, from center to center. These hangers are of the Ohio Brass Company type, and are attached to the messenger cable with a sister hook through the base of which the rod is threaded and drawn up tight against the messenger cable. The 0000 grooved trolley is secured to the hanger by Detroit clamps.

At each bridge there is a span-wire steady strain (not shown in the photograph), the trolleys being insulated from the bridge and from each other by 6-in. wheel-type porcelain strain insulators. The messenger cable is dead-ended on an equalizer attached to a pair of these insulators, which are connected by short cable loops to a similar pair secured to the anchor bridge.

A test which came on the line during construction showed the effect of a broken messenger wire. When about half a mile of wire had been pulled up, and hangers were on three or four of the spans, the dead-end arrangement broke, allowing the line to go. The insulator broke on the bridge next to the dead-end bridge, but the trouble did not extend any further than that point. Some of the men on the work thought the foundation of the second bridge was raised a little, but there seems to be considerable difference of opinion on that point, and certainly no damage was done. The idea has been that the bridges would be what might be called semi-anchored; or, in other words, the effect of a break would not go beyond two or three bridges in either direction. It is possible, too, that the heavy rods and tight clamping effect, obtained both at the messenger and trolley wires, assisted considerably in holding the line. At any rate, no damage was done which could not be repaired quickly.

The high-tension wire is strung with a sag of 40 in. at 20° fahr., and has a net clearance of about 24 ft. from the track. As the trolley may at some time be operated at 6 600 volts, single-phase, all the insulation of the catenary, steady strains, etc., is designed to withstand this voltage. At present, however, it is being operated at 600 volts, direct-current. In regard to the lateral stiffness of the single catenary with supports 300 ft. apart, it may be noted that the line has been in operation about 10 days, and, thus far, very little deflection or rolling can be observed under the action of the wheel trolley with a tension of about 25 lb. During this time there have been temperature changes of 50° and wind velocities of 45 miles per hour. These conditions seemed to make no difference in the operation, even before the steady strains were installed.

The speaker feels justified in saying positively that, with this type of construction, there will be no difficulty from side-sway on the 300-ft. span. It is yet to be determined whether the line is too stiff in the vertical plane, but that cannot be determined positively except by operation extending over a considerable period, and including hot as well as cold weather. The preliminary tests which have been made indicate that there will be no difficulty.

SINGLE CATENARY CONSTRUCTION, SYRACUSE, LAKE SHORE AND NORTHERN RAILROAD.

This line is a re-location, to shorten the running time and provide a double track on private right of way between Syracuse and Baldwinsville. The old single-track line is on a highway, and is about ¾ mile longer than the new location, which will form part of a new high-speed electric road between Syracuse, Fulton, and Oswego, a total distance of about 35 miles. Mr. Archbold.

CHARLES RUFUS HARTE, M. AM. SOC. C. E.—In the field of transmission- and distribution-line construction, each engineer has been largely a law unto himself, and Mr. Coombs' effort to secure some measure of standardization is much to be commended. At the same time, local conditions very largely govern, and the successful construction of one locality may be of little value even in comparatively near sections. Mr. Harte.

In addition to the causes given by the author, a short circuit may be caused by the swaying together of two phases of the circuit. This, however, may be prevented by spacing the wires a distance apart equal to at least twice the versed sine of the sag. Thus the Missouri River transmission has a spacing of 78 in.; the 1 450-ft. Connecticut River span of the Springfield-Suffield Line, 84 in.; while the Madison River Line, of Montana, has 108 in. As an additional precaution, the wires are often arranged so that no two are in the same horizontal plane. This is characteristic of the Connecticut River span, the Anglo-Mexican, the Southern Power, and many other transmission lines. The large spacing (of the triangle), with two wires in a vertical plane, also materially reduces the likelihood of interference from large birds, or branches or wires blown against the line. In practice, however, because of their weight and low periodicity, long spans usually swing in unison, thus maintaining the spacings.

Mr. Coombs states that insulator troubles are largely due to misdirected savings. While it is true that there are to-day many lines operating without change at higher voltages than designed for, and on which the insulators were poor for the original voltage, there are many other lines where, although no expense has been spared, insulator troubles are very serious.

On the seacoast, particularly in Southern California, heavy salt fogs cause troubles which, as far as the speaker knows, have not yet been overcome successfully; in the alkali deserts, the so-called salt storms result in losses by very remarkable brush discharges and leakages; and where lines are near steam-railroad right of way the oil and water from the exhaust condense on the insulators and then collect coal and other dust until the creeping surface is largely covered, causing heavy leakage, and burning wooden pins. This condition threatens to be a very serious problem in steam-road partial electrification. The deposits from salt and dust storms are washed off by the rains, but the oily coating resulting from locomotive exhausts is not affected by water.

Mr. Harte. While a "campaign of education" may be of assistance, the small boy with his sling-shot and the man with the gun will always menace seriously the welfare of insulators in settled sections. Dark-colored glazes, being less conspicuous, are being used in many cases. With medium voltages, compact insulators of the Redlands or Crown type are used, and one large manufacturing company grooves the insulator top, with the idea that the marksman will knock out the portion inside the groove and then retire satisfied, leaving enough insulator on the pin to protect the line. As a matter of fact, against a bullet of any weight there is little choice as to type. The speaker tested the three types shown, Fig. 1, Plate LXV, using a Winchester $\frac{38}{100}$-caliber rifle, reproducing line conditions as far as possible, and firing one shot at each. Fig. 2, Plate LXV, shows the result. In New England or the East generally, a gun of such heavy caliber would rarely, if ever, be used.

Serious sleet storms, fortunately, are not common, and are rarely of great extent. While failure may be due to the dead weight of the accumulation, the usual cause is more complex. In a strong wind, the sleeted wires, because of the greatly increased area and weight, sway until the vibrations become synchronous with the natural period of a pole of the line. If this pole fails, the resulting long span usually has weight enough to pull down other poles on each side.

Aluminum does not appear to hold sleet as does copper, and lines transmitting much power are usually a little warmer than the air, so that they throw off the coat quickly; therefore, an allowance of $\frac{1}{2}$ in. of ice around the wire is sufficient to provide for all reasonable contingencies. The sleet coatings, however, may become very heavy. Fig. 1, Plate LXVI, shows an accretion of clear ice on a twig, the coating having a diameter of practically 3 in.; and C. J. H. Woodbury, M. Am. Soc. C. E., recently advised the speaker of specimens he had seen, one from Wayland, Mass., $4\frac{1}{2}$ in. in diameter, and one from Nahant, 8 in. in diameter. Fig. 2, Plate LXVI, shows the accumulation on wires caused by the same storm which developed the twig coating. This was at Winsted, Conn., on February 21st, 1898.

Occasionally, sleet and snow storms make trouble by bridging over the creeping surface of the insulator, thus causing leakage to the cross-arm. This, however, can be prevented by designing the pins and cross-arms so as to leave no large catchment area, and by maintaining a considerable distance from the insulator top to the arm.

The pressure variations in wind storms, cited by Mr. Coombs as a reason for using a low value, are more properly reasons for introducing a sway factor. A field of grain or long grass, or ivy on a house, observed in a wind storm, shows very clearly the successive pressure waves. On a transmission line, when such impulses are synchronous

FIG. 1.—INSULATORS, BEFORE THE TEST.

FIG. 2.—INSULATORS, AFTER THE TEST.

with the natural period of one of the poles, stresses are set up in the latter far in excess of those due to the direct forces themselves. In wooden pole lines there are stress transfers, due to the elasticity of the poles and the slipping of the wires at the insulators, which relieve these unusual conditions; in tower lines, the greater rigidity of construction largely prevents such relief, and the action must be considered in designing. Mr. Harte.

Mr. Coombs gives a series of very valuable tables of wire factors, but there is a matter in connection with solid copper that, as far as the speaker is aware, has been given practically no attention by any investigator. The standard American copper-wire bar weighs approximately 200 lb. This is rolled to a rod of diameter depending on the gauge of the wire it is to make. For 0000 trolley, this rod is about 560 mils in diameter, and is less than 300 ft. long. To secure the long commercial lengths of trolley, the rods are brazed together, the scarf having an angle of about 20°, the brazing being done with a mixture of silver and tin at a temperature near 800° fahr. As a result, the rod is annealed at the scarf, and the subsequent drawings to a diameter of 460 mils do not harden this annealed portion.

From tests of brazes, it appears that their strength is only eight-tenths of that of stock wire. Incidentally, it should be noted that the strength of grooved 0000 wire is from 4 to 5% lower than that of round wire, due to the fact that more work is done upon the latter. Grooved wire cannot be given a second reduction after the groove has been made.

The failure at a braze does not occur in the braze itself, but in the area immediately adjoining; the break is usually parallel to the scarf, but there is invariably a skin of copper on the braze.

At least one wire manufacturer uses a specially heavy wire bar when requested, thus having fewer brazes per mile, but, in any case, it is the braze which determines the strength of the line. For this reason, as well as because of its flexibility and consequent ease of handling, stranded copper is much better than solid for transmission-line work, the strand brazes being distributed along the made-up cable. It should be added that the problem of the braze is now occupying the attention of a number of the large wire manufacturers, and it is hoped that decided improvements will follow.

Mr. Coombs refers to a grooved trolley wire having an area of 0.155 sq. in. and an ultimate strength of 8 800 lb. The speaker would like to have further details of this wire. Commercial, American, 0000, grooved wire, having a cross-section of 0.167 sq. in., this being a little more than 7% greater than the wire referred to, in a series of tests, failed to reach an ultimate strength of 8 000 lb., the break usually occurring at about 7 800 lb.

Mr. Coombs' type of anchor is good for comparatively light

Mr. Harte. stresses, but for heavy spans it is desirable to arrange the insulators in pairs, or, if in tandem, to the catenary of the span, to secure uniform stress distribution.

TABLE 11.—Tests of 0000 Grooved Trolley Wire.

Length tested in each case, 10 in.

Tests of Brazed Joints.

Test number.	Height of cross-section at braze, in inches.	Length of joint, in inches.	Breaking strain, in pounds.	Percentage of elongation, in 10 in.	Remarks.
3 629	0.475	1.4	5 852	1.5	Apparently good braze. Broke in joint, with partial separation of braze.
3 630	0.476	1.5	6 306	6.7	Apparently good braze. Broke outside joint.
3 631	0.448	0.9	4 429	14.7	Apparently good braze. Metal reduced in section by filing. Broke in joint, with partial separation of copper.
3 632	0.475	1.4	6 175	6.5	Apparently good braze, broke outside joint.
3 633	0.452	1.1	4 515	13.9	Apparently poor braze. Metal reduced in section by filing. Broke in joint, with partial separation of copper.
3 634	0.476	1.3	6 390	3.8	Apparently good braze. Broke in joint. Separation very slight. Flaw in copper at point of rupture.
Average.	0.467	1.3	5 611	7.9	

Tests of Wire From Sections Between Joints.

3 628	0.478	....	7 087	7.0	Fracture silky, angular.
3 635	0.478	....	7 085	5.7	" " "
3 636	0.478	....	7 166	5.4	" " "
3 637	0.478	....	7 077	8.3	" " "
3 638	0.478	....	7 076	5.9	" " "
Average.	0.478	....	7 098	6.5	" " "

Where the sag must not fall below fixed limits, provision must be made for adjustments of considerable extent. In Mr. Coombs' design, any considerable take-up on the turn-buckles would result in slack on the saddle, not readily cared for. This may be avoided by using a double saddle, both parts being movable, the slack looping between. The Connecticut River crossing of the Springfield-Suffield line has a crossing span attached to a movable cross-head controlled by a long screw with an adjusting nut, Fig. 9. The main line taps into the crossing span at the cross-head, and has a "pigtail" to care for the variation in length.

FIG. 1.—SLEET ACCRETION ON TWIG, WINSTED, CONN., FEBRUARY 21ST, 1898.

FIG. 2.—SLEET ACCRETION ON WIRES, WINSTED, CONN., FEBRUARY 21ST, 1898.

Under specifications, to bar thin wiped galvanizing, it is desirable to require the galvanized metal to stand four immersions, of 60 sec. each, in a saturated solution of copper sulphate, at 70° fahr. After each immersion the test piece should be dipped into clean water and then wiped dry; no metallic copper should appear after the fourth immersion. Mr. Harte.

The speaker wishes Mr. Coombs had treated the subject of protection of line crossings at greater length. Apparently, the crossing described is protected, over and above the general line, only by taking additional precautions to relieve the crossing towers from stresses due to adjoining spans, and in certain details of anchorage of line, provision being made for the installation of a cradle at a later date if desired.

As far as line strength is concerned, a crossing differs from a normal span only in possible greater length, or in restrictions as to height of wire; and a failure at this point, in its effect on the service, does not differ from a failure elsewhere; but, in the possibilities of damage to train, to passengers or others on platforms or highways, or to other lines, the crossing becomes one of the most critical line points, and the method of safeguarding it is of the utmost importance.

Protection may be effected by:

1.—Mechanically preventing a broken wire from getting into the danger section;
2.—Strengthening the upper wires so that failure is practically impossible;
3.—Grounded arms to cut off a broken wire at the pole top before it can reach the line below.

Of the first class are the various cradles, all open to the criticism that they offer large areas for sleet lodgment, and most of them that they are very expensive.

A very simple type consists of telegraph wires strung between multiple pin arms. The system is grounded, but the small section of the wires is usually a guaranty that they would be burned through by the arc in case of a fall of the power line. Fig. 1, Plate LXVII.

A modification consists of three longitudinal wires with cross-bars of hard wood; other variations include sheets of wire netting, networks of wire strand more or less substantially fastened together, up to the very impressive cradles of heavy strand with cross-bars of flat iron. As clearly appears in the case shown, Fig. 1, Plate LXVIII, wood bar cradles are apt to lose members from breakage or otherwise, while the heavier wire cradles often sag, becoming a positive menace. In Fig. 2, Plate LXVII, is shown a wire-strand cradle which has sagged to an extent requiring the power company to protect its lines by the support wires strung on the top arm of the transmission line.

Mr. Harte. While the more substantial types, if large enough to keep a fallen wire from blowing out again, are no doubt efficient along certain lines, their great cost, and the excessive stress imposed by them upon their supports, even without the great loads of sleet they are sure to catch, make them very undesirable. Fig. 1, Plate LXIX, shows a recent installation at Coltsville, Mass., of a grounded strand-wire net, in accordance with the latest specification of the American Telegraph and Telephone Company for this type of protection. The stresses on the poles and the high cost of the construction are obvious.

The ideal protection is the so-called short-span method. Here the crossing span and the two spans adjoining are arranged so that the distance apart of the poles is less than the distance from the cross-arm of the upper line to the lower line; it is thus impossible for the two lines to touch, under any circumstances, while the adjoining short spans prevent a broken wire from swinging into the crossing-span section. If the adjacent spans cannot be made short, a grounded guard arm may be used to effect the same purposes. Fig. 2, Plate LXIX, shows a typical installation at Pittsfield, Mass., having on the right such a grounded guard arm with vertical wings to catch a broken wire which might otherwise be blown clear of the guard, while the left side is short-spanned, and is further protected by the angle in the line at this point. Unfortunately, crossings usually occur in highways where limitations as to pole locations prevent the use of this method.

The method most generally applicable, and, in the speaker's judgment, the best, is that of reinforcing the line by a set of messenger cables. This plan has the great advantage that the line is locally doubled in strength, with but little increase in weight or in exposure area, and the cost is nominal.

In a recent and satisfactory design, messengers of No. 2 stranded copper are used, and to them the line wire is tied every 4 ft. A grounded angle-iron frame just beneath the line provides an automatic cut-off in case the line breaks in the span adjoining the protected span, Fig. 10, and Fig. 2, Plate LXVIII.

A protection device should meet the following requirements:

1.—Complete protection of the lower line, including protection from a failure in an adjoining span with wires whipping into the protected section;
2.—A minimum of areas exposed to sleet or wind;
3.—Little increase over weight of normal line;
4.—Simplicity of design;
5.—Construction familiar to linemen;
6.—Few special parts;
7.—Low cost of installation and maintenance.

The foregoing types of protection assume that the power line is above, and the additional safety of such arrangement will usually

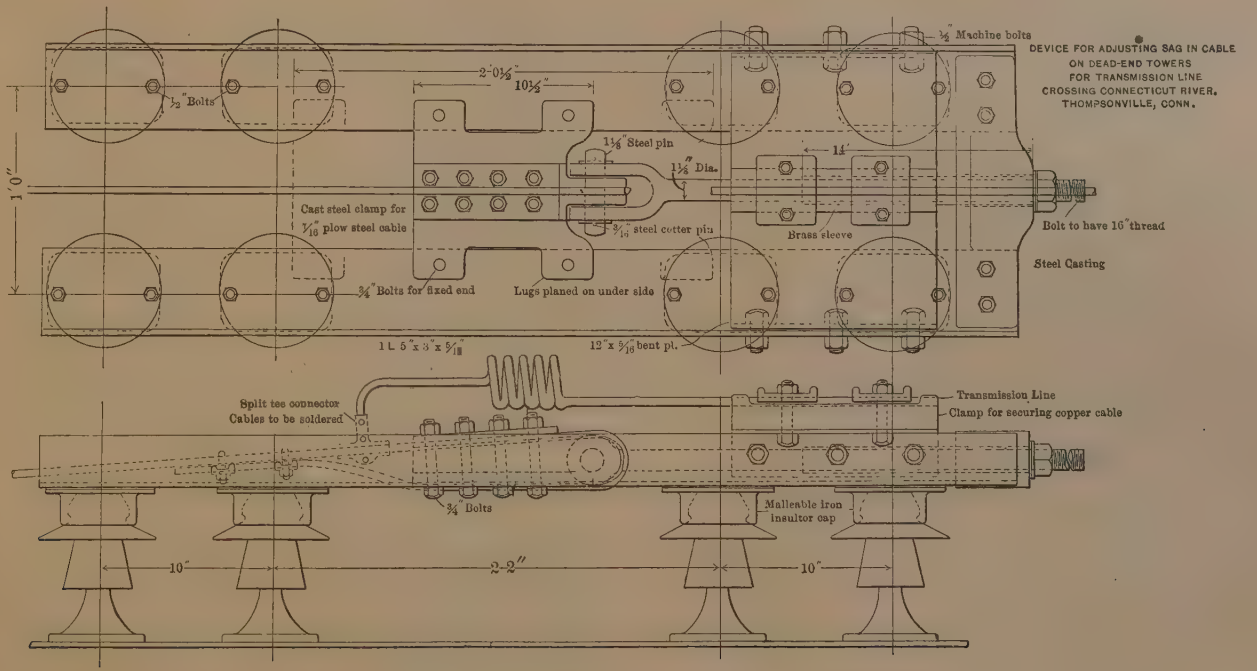

DEVICE FOR ADJUSTING SAG IN CABLE
ON DEAD-END TOWERS
FOR TRANSMISSION LINE
CROSSING CONNECTICUT RIVER.
THOMPSONVILLE, CONN.
½" Machine bolts
½" Bolts
2'-0½"
10½"
1'0"
1⅛" Steel pin
1⅛" Dia.
14"
Cast steel clamp for
7/16" plow steel cable
⅜" steel cotter pin
Brass sleeve
Bolt to have 16" thread
Steel Casting
¾" Bolts for fixed end
Lugs planed on under side
1 L 5"x 3"x 5/16"
12"x 5/16" bent pl.
Split tee connector
Cables to be soldered
Transmission Line
Clamp for securing copper cable
¾ Bolts
Malleable iron
insultor cap
10"
2-2"
10"

FIG. 9.

Mr. Harte. justify a considerable expenditure to secure it. In some cases, however, it is practically impossible to go above with the transmission line. In such cases the method of the American Telephone and Telegraph Company, of practically enclosing the upper line in a sheath of $\frac{3}{8}$-in. strand network is the best, provided the design is such that it will prevent dangerous sagging of the cradle.

In any cradle design, the tendency of a broken wire to curl and therefore jump out of the cradle should be recognized; in Germany and Switzerland it is customary to compel transmission companies to make all lines crossing railroads pass through a regular tunnel of ironwork.

Where the line is on very narrow right of way, and where it carries trolley brackets, straight poles are essential, and it is often desirable to use selected stock in important highways, but in the majority of cases considerable crook can be allowed. Certainly, where chestnut is to be used, Mr. Coombs' requirement of only 1 in. of crook in 10 ft. of length is unnecessarily rigid.

The Western Lumberman's and the Idaho Cedarmen's Associations have defined commercially straight cedar poles as having a crook in one direction only, and a sweep not to exceed 1 in. in 6 ft. For chestnut, the American Telephone and Telegraph Company allows practically 1 in. sweep in $2\frac{1}{2}$ ft. of length for poles up to 40 ft. total length, and of 1 in. sweep in 3 ft. for poles more than 40 ft. in total length, the measurements to be made between the top and a point 6 ft. from the butt.

Chestnut from seed often grows very straight; stump-grown stock, which to-day forms a large proportion of the supply, almost invariably shows a sharp crook near the butt, due to the growth of the shoots, first out to clear each other and then straight upward. If this crook is large, it increases the cost of pole setting, but a divergence from the general axis of the pole of not more than 12 in. in the lower 6 ft. can be cared for without additional work.

In the speaker's judgment, the severity of the specification does not increase the line strength, and, as it materially increases the cost, it would seem that it might better be changed, to meet current practice and market limitations, to the following:

Cedar poles shall have but one crook, this in one way only, the sweep not to exceed 1 in. in 6 ft.

Chestnut poles shall have but one crook, this in one way only. The sweep shall not exceed the following limits between butt and top:

Pole length, in feet:	30	35	40	45	50	55	60	65	70.
Sweep, in inches:	9	10	11	11	11	12	13	14	15.

As far as the speaker is aware, there has as yet been no wreck of any magnitude on any of the electrified steam lines, and, until such a try-out, certain questions of design must remain unanswered.

FIG. 1.—WIRE GRIDIRON UNDER 33 000-VOLT TRANSMISSION LINE.

FIG. 2.—WIRE CRADLE OVER 11 000-VOLT TRANSMISSION LINE.

Mr. Harte.

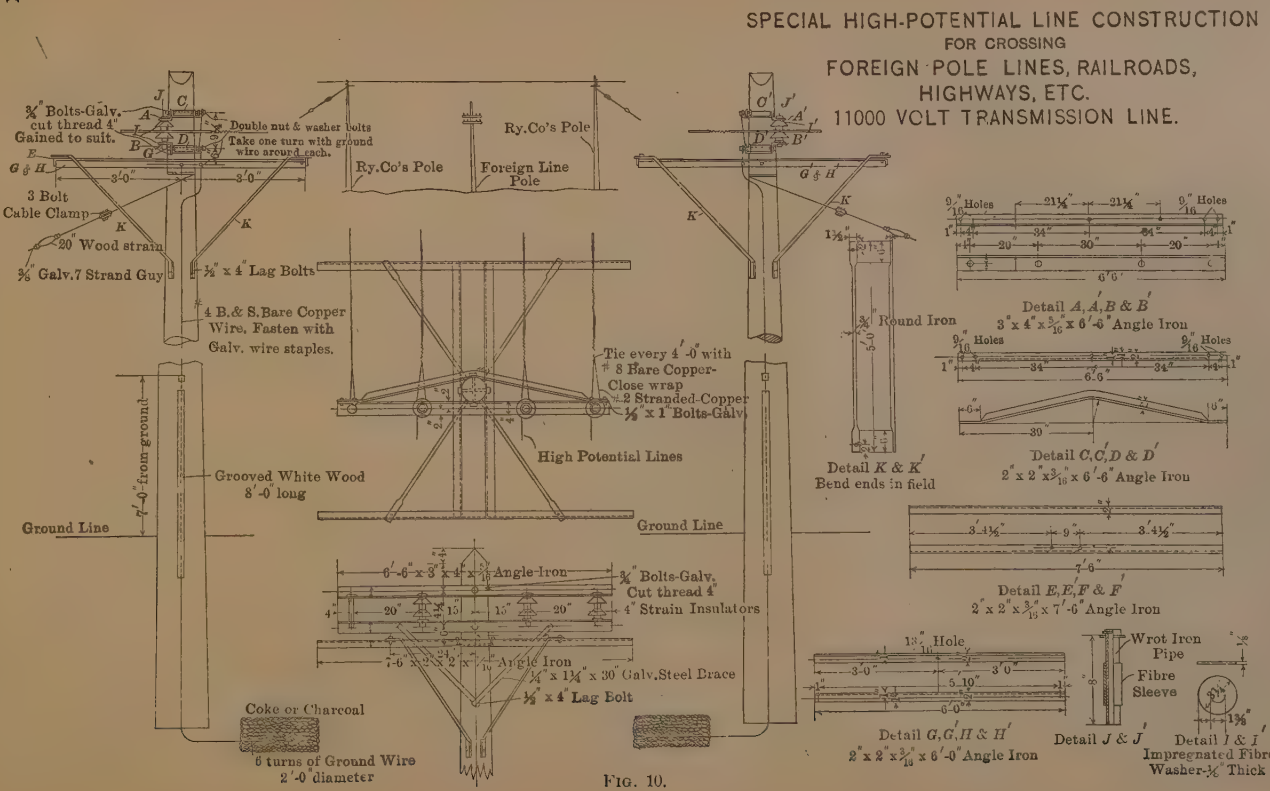

SPECIAL HIGH-POTENTIAL LINE CONSTRUCTION
FOR CROSSING
FOREIGN POLE LINES, RAILROADS, HIGHWAYS, ETC.
11000 VOLT TRANSMISSION LINE.
¾" Bolts-Galv. cut thread 4" Gained to suit.
Double nut & washer bolts
Take one turn with ground wire around each.
Ry.Co's Pole
Ry.Co's Pole
Foreign Line Pole
3 Bolt Cable Clamp
20" Wood strain
⅜" Galv.7 Strand Guy
½" x 4" Lag Bolts
4 B.& S. Bare Copper Wire, Fasten with Galv. wire staples.
Tie every 4'-0" with # 8 Bare Copper-Close wrap #2 Stranded Copper
⅛" x 1" Bolts-Galv.
High Potential Lines
Grooved White Wood 8'-0" long
7'-0" from ground
Ground Line
Ground Line
6'-6" x 3" x 4" x 5/16" Angle Iron
¾" Bolts-Galv. Cut thread 4"
4" Strain Insulators
7'-6" x 2" x 2" x 1/10" Angle Iron
¼" x 1¼" x 30" Galv. Steel Brace
½" x 4" Lag Bolt
Coke or Charcoal
6 turns of Ground Wire 2'-0" diameter
1½"
¾" Round Iron
5'-0"
Detail K & K' Bend ends in field
9/16" Holes
21¼"
30"
6'6"
Detail A, A', B & B' 3" x 4" x 5/16" x 6'-6" Angle Iron
Detail C, C', D & D' 2" x 2" x 3/16" x 6'-6" Angle Iron
Detail E, E', F & F' 2" x 2" x 3/16" x 7'-6" Angle Iron
13/16" Hole
Detail G, G', H & H' 2" x 2" x 3/16" x 6'-0" Angle Iron
Wrot Iron Pipe
Fibre Sleeve
Detail J & J'
Detail I & I' Impregnated Fibre Washer-½" Thick

FIG. 10.

Mr. Harte. It is not at all unlikely, however, that, at least for lines on which freight trains, with their capacity for trouble, are handled, the ultimate development in steam railroad electrification will be in the direction of independent overhead lines for each track or group of tracks.

With one live trolley, emergency movements can be made on adjoining tracks; with all overhead wires down, as may well be feared in a wreck under bridge construction, not only is the electrical equipment helpless, but a large additional burden of clearing away the material devolves on the wrecker.

Whatever the design, the speaker feels that the unit stresses allowed by Mr. Coombs are too high. With long and frequent trains, and particularly with high voltages, the failure of any part of the overhead system offers too great an opportunity for serious results to justify any close paring in the design.

Steam railroad electrification for some years to come will be undertaken only where there is in sight a very marked gain by the change, or where legislation compels it. The complications with which the simplest distribution system involves maintenance operations, and the awkward fact that for wrecking, and in "dead" sections, some self-contained motor must be used, weigh heavily with men familiar with steam-road operation, and offset many of the obvious advantages of electrification. It will rarely happen that a cost variation several times in excess of the difference between thoroughly dependable construction and "probably safe" construction will be of weight in influencing the decision, and in the few cases where it is a factor it is far better for the art that the work be deferred rather than incur an unjust discredit because of failure, either physical or in performance, as to expected maintenance and operation costs.

Whatever the unit stresses, the bridges, brackets, or poles should have a safety factor considerably in excess of that of the overhead system proper, and messengers, hangers, and trolley should mark a regularly descending scale, in order that any failure may be of the least extent possible.

While it is true that the ordinary trolley suspension gives a catenary curve, the general practice to-day is to apply the term "catenary construction" to systems supporting the trolley from one or more messengers. Following this practice, overhead systems would be classified as:

1.—Simple Suspension.—Trolley carried by hangers directly connected to span wire, bracket, or bridge;

2.—Catenary Suspension.—Trolley hung from one or more messenger cables in turn carried by the span wires, brackets, or bridges;

3.—Single Catenary.—Having but one main messenger (as in the Erie Railroad electrification);

FIG. 1.—WOOD-BAR CRADLE UNDER 6 600-VOLT LIGHTING CIRCUIT.

FIG. 2.—CATENARY CROSSING, WITH SINGLE CUT-OFF ARMS.

Mr. Harte.

4.—Multiple Catenary.—Having more than one main messenger (as in the New Haven electrification);

5.—Simple Catenary.—Trolley carried directly by main messenger; may be simple or multiple (as in the Erie or New Haven electrification);

6.—Compound Catenary.—Trolley carried by a secondary messenger system, in turn carried by the main messenger (as in the Blankenese-Ohlsdorf Railway).

The kind of conductors best adapted to the collection of the power is an open question. The two chief difficulties, with high speeds, are the chattering of the shoe, due to alternate hard and soft spots in the line, and the pressure variations, due to the great vertical range required of the pantograph.

The first problem may be solved either by floating or by fixing the trolley wire; for undoubtedly a perfectly flexible line or one perfectly rigid would give excellent results as far as it alone was concerned. Whether the shoe will not chatter on the rigid line, as a result of the irregular movements of the car, remains to be seen. It is interesting to note that Mr. W. S. Murray, Electrical Engineer of the New York, New Haven, and Hartford Railroad, in discussing his recent paper before the American Institute of Electrical Engineers, is quoted* as saying that, in his judgment, either the shoe or the line must be flexible.

The second problem is largely a function of overhead crossing limitations, and, to a large degree, is independent of the overhead construction; therefore it must be cared for in the design of the collector itself.

Both problems are of the field rather than of the office. Mr. Mayer† has given a very elegant mathematical analysis of shoe pressure under certain conditions, but the discussion is based on the supposition that the car end of the collector traverses a path bearing a definite and regular relation to the conductor. As a matter of fact, however, this path is most irregular. Unevenness of track, as to grade and line, gauge variations of rail and wheel, side play in axle boxes, spring action, and movements in the car framing itself, all affect the shoe pressure entirely independently of the variations due to the collector mechanism and the character of the overhead system.

Mr. Coombs recites five objections to the double, as compared with the single, catenary. That the double catenary has greater first cost and greater mass overhead is true, although, by the time the single form has been properly secured by pull-offs, guys, and steady braces, there is a surprising amount of material in the air.

As to maintenance, however, the speaker doubts whether a single

* *Street Railway Journal*, January 18th, 1908, page 81.
† *Proceedings*, Am. Soc. C. E., for December, 1907.

Mr. Harte. catenary is not at least as troublesome. A double catenary can stand severe punishment and still permit the movement of trains. On the other hand, the hangers of the single catenary are more out of the way, and therefore less likely to be injured.

Either type requires the tower car for repairs, but the double catenary has twice as many connections to make; on the other hand, its greater strength and rigidity undoubtedly reduce the troubles above the trolley.

That the double catenary offers greater obstruction to the view of the signals, the speaker cannot admit. If the signals are on bridges, they will be between the tracks, and a curve that would bring the overhead structure across the line of sight would also bring the pole, towers, or truss posts also into line. The difficulty relates to the secondary supports rather than to the type of suspension.

Mr. Coombs sums up the situation admirably. If anything remains to be said, it is this: In the present state of the art there is a great lack of, and need for, data resulting from practical tests of the various theories.

In closing, the speaker wishes to express his obligations to the many friends who have kindly assisted in the experiments, and have loaned illustrations for use in this discussion.

Mr. Osgood. FARLEY OSGOOD, ESQ.* (by letter).—If the high-tension wires are of sufficient mechanical strength to have a factor of safety of 3, under correctly-assumed general conditions, it is very doubtful if a conductor will part in the span.

Up to crossings of 600 ft., it is not considered that the wires are likely to cross in high winds, even though spreaders are not used, as experience seems to indicate that the wires will swing from their normal positions about equally.

Protection, in the form of lightning rods, seems desirable at crossings where very high wooden towers are used, or on lower wooden crossings at points of high altitude.

If steel towers are used at railroad crossings, the use of lightning rods is desirable, if the crossings are at such points in the line as are known to be affected by lightning disturbance.

The use of cradles, suspended from high-tension poles, under the high-tension wires, is not advocated by the writer, for any ordinary circumstances.

An ideal high-tension crossing would have the supporting towers of sufficient height to make it impossible for one of the transmission wires to touch the ground in case it should break in the crossing span, but this condition is usually impossible, from a rational standpoint, owing to the length of the section.

* General Superintendent of Distribution, of the Public Service Corporation of New Jersey.

FIG. 1.—CRADLE PROTECTION. GROUNDED NET OF STRANDED WIRE.

FIG. 2.—SHORT-SPAN PROTECTION, WITH GROUNDED GUARD ARM.

A second choice for crossings seems to favor a supporting tower at the edge of the railroad company's right of way, and another between the telegraph or signal wires and the outside rail, thus using four poles or towers to a crossing, making a short span on each side of the tracks over the telegraph or signal wires and the longer span over the tracks, it being assumed that the railroad company has wires to be protected on each side of its tracks. Mr. Osgood.

These poles should be of sufficient height to prevent one of the high-tension conductors from touching the telegraph or signal wires, if it should break, so that, if a high-tension conductor should give way over the tracks, the signaling system would not be affected, although this might not always be true if any of the rails were used as part of the signaling circuit.

If railroad companies feel that screens or cradles should be placed under high-tension wires, let such devices be placed in the top arm positions on the poles carrying the telegraph or signal wires, as their purpose can then be accomplished and no unnecessary burden be placed on the more important high-tension towers.

A simple and inexpensive type of screen, if used as suggested, can be made up as follows: In the top gain of the pole on each side of the high-tension crossing, place a ten-pin cross-arm, somewhat longer than the standard cross-arm used on the line to be protected, and run in between these arms, ten No. 6 or No. 8 galvanized steel wires, strapped together, and grounded at each pole.

W. S. Murray, Esq.* (by letter).—Mr. Coombs has handled this very interesting subject in an analytical and conservative manner. With reference to that part of his paper concerning the strength of the materials to be used for wires and supporting structures, the basis of his assumptions could not be better founded than on the records given in his several tables. With reference to the equations relating to the unit pressures per square foot of projected area, the writer is pleased to be able to confirm these figures in actual practice, as the catenary wires and supporting structures in the New Haven electrification were worked out by an equation practically identical with the one suggested by Mr. Coombs; and it is of interest to note here that these wires and structures have passed through storms approximating quite closely those stated as maximum conditions upon which the equations are based. Mr. Murray.

Mr. Coombs' specifications of general requirements are to the point, and, in addition to those which are generally recognized as standard, he has made many original and valuable suggestions.

In connection with the general subdivision of superstructures, although the writer is not quite able to agree with Mr. Coombs that the upright signals when supported from four-track trusses are obscured

* Electrical Engineer, New York, New Haven and Hartford Railroad.

Mr. Murray. from the engineer's view at a distance of 1 200 ft., it is unquestionably true that the general envelope produces a difficult foreground for the engineer, and naturally the cross-span or cantilever-bracket construction clears up this disadvantage to a considerable degree.

As recently stated in a paper before the American Institute of Electrical Engineers, the writer is not loathe to believe that even four-track main-line electrification will be effected by the use of cross-catenary spans interspersed at proper intervals with fabricated steel truss anchor bridges; but believes that the form of this construction will be guyed steel uprights supporting the cross-catenary span, with distances between bents of, say, not greater than 300 ft.; and, further, he believes that, in the future, the single catenary will receive more favorable consideration than the double catenary construction. It can be readily seen that the first cost of the former will be much less, and the flexible contact offered by the single catenary construction, due to the fact that the trolley is supported from a single messenger, with the messenger in turn supported from a flexible cross-catenary, gives it a great advantage.

Practice seems to demonstrate the fact that either the shoe or the trolley must be flexible. As a matter of fact, flexibility in both would be of great advantage, and it cannot be questioned that the cross-catenary span will offer more flexibility than either the cantilever or bridge-truss type of construction. At this point, particular attention is called to the fact that experimentation with the deflection of trolley wire supported from a messenger, which is in turn supported at rigid points, shows that in the middle of the span the deflection is as much as 400% greater than that in the immediate vicinity of the bridge or cantilever supporting the messenger wire, it being understood, of course, that equal upward pressures are applied in each instance. This illustrates the value of the flexible feature in the cross-catenary support. A point of much value in the cross-catenary construction should be emphasized, namely, that the cross-spans may be supported on strain insulators, thereby not only doubling the actual insulating value of the line, as measured under normal atmospheric conditions, but, in point of fact, many times increasing the insulating value due to the insulation being placed at the side, and thus out of the direct line of steam locomotive blasts, which have such a deleterious effect on insulation.

An argument that will be advanced against the use of the cross-catenary construction is that it is not as reliable as the cantilever or truss construction. The answer to this is that, in this form of construction, any factor of safety that may be used in other types can be selected; in fact, larger factors of safety can be chosen with less proportionate expense.

In conclusion, the writer agrees with Mr. Coombs in his summation, under five counts, concerning the undesirability of double cate-

naries. The root of all trouble with the alignment of catenary construction is the change of temperature. The fact that a low temperature means a tight wire and *vice versa* for a high temperature must be considered. The ideal condition of suspension would be a free-running suspended wire, tension being supplied at one or both ends to counteract the variations in its length due to temperature. It is very seldom that ideal conditions can be secured in the field, however, and the results are generally a combination of compromises and approximations. What one fails to accomplish with the contact wire may be accomplished by a properly devised shoe, of strong construction, flexible and light, the last-named element eliminating inertia, the arch enemy to the hard spots in the line, which, as Mr. Coombs has pointed out, are at the "hanger points." To-day is not the time for standardization, but for observation. The experiences and mistakes of to-day will be invaluable in comparison with theories. Mr. Murray.

R. D. Coombs, M. Am. Soc. C. E. (by letter).—In presenting this paper, the writer hoped that a discussion might be elicited that would lead to a construction more rational than some now in existence. In illustration: A certain high-tension power line crosses a double-track branch road by a bridge, the posts of which are outside of a wide right of way, and on which the wires are supported at approximately 10-ft. intervals; in another case, the power line is supported by wooden poles, and the construction is about comparable to ordinary telephone practice. Fig. 1, Plate LXX, shows a guard bridge which is both expensive and unsightly. In order that the development of electric transmission may not be unduly burdened by conservatism, or be subject to stringent legislative action, as the result of avoidable accidents, it is to be hoped that future construction may be upon a more consistent basis. Mr. Coombs.

The differences in loading between the crossing specifications and the traction specifications were due to probable differences in location, coupled with the fact that the latter class of construction would be subjected to closer supervision by the railroad company's engineers. Considering the same general territory, the writer has revised the specifications so that the conditions of loading are identical, and, for the Eastern States, he has used a maximum loading of ½ in. thickness of ice and a wind load of 8 lb. per sq. ft.

In view of the increasing cost of timber, it will no doubt be economical to use creosote or other preservative on wooden poles, and avoid the greater expense of concrete foundations.

Unless the towers or poles supporting high-tension wires are on private right of way, some form of danger notice may be desirable, and, in case ladders are provided, the lower section should be hinged and kept raised, or omitted altogether.

The originals of the two types of anchor insulators, shown on

Mr. Coombs. Plate LXXI and Fig. 11, were probably devised by Ralph D. Mershon, M. Am. Soc. C. E., and the writer understands that both the original and revised types have given satisfactory service.

Mr. Harte's criticism of these anchorages is entirely correct, for the conditions he outlines, but is not usually applicable to the short spans and slightly variable sags of railroad crossings.

In the original specifications, mention was made of the possible addition of cradles, though, in the writer's opinion, cradles are undesirable unless supported by independent poles. The short-span method of protection, cited by Mr. Harte and by Mr. Osgood, is very rarely feasible on account of local conditions, and in crossings over electric-traction railroads would make extremely high poles necessary.

It is probable that placing a cradle or enclosing screen on the telephone line would be economical in some cases, and would usually provide greater security as far as the telephone line is concerned. The specification requirements would still be applicable to the power line, as such protection may not prevent disarrangement of the signal circuits by falling power wires.

Referring to the three classes of protection mentioned by Mr. Harte, the writer's specifications were intended to provide the strength necessary to prevent failure, while allowing the installation of such mechanical and secondary structures as permitted by local conditions.

The seven requirements of a protection device enumerated by Mr. Harte may all be said to be included in his first and seventh requirements. Complete protection combined with low cost is not attained by any arrangement of which the writer is informed.

Supporting catenaries for power-wire crossings have been used to some extent west of Pittsburg, Pa., but the writer cannot agree with Mr. Harte that their use involves little additional cost, because for each power wire there are required a separate messenger and a large number of short hangers, and the weight and the area exposed to wind pressure are about doubled. To apply such a requirement to a large power line, as, for example, 24 wires of 250 000 cir. mils section, would involve extremely heavy and unsightly construction.

The writer does not apprehend failures on account of the pressure variations in wind storms. As originally stated, such variations reduce the average load upon the wires of long spans to less than some of the recorded values. In regard to the effect of such impulses upon the poles, it seems improbable that the difference between the sums of the impulses in adjoining spans, on which the longitudinal movement of the pole depends, can be synchronous with the motion of a pole when the latter is restrained by catenaries.

For the third condition of loading, in the revised specification, a slight deflection of the pole top will relieve the unbalanced load by altering the sag of the wires, and the first condition will generally be the governing one.

Mr. Coombs.

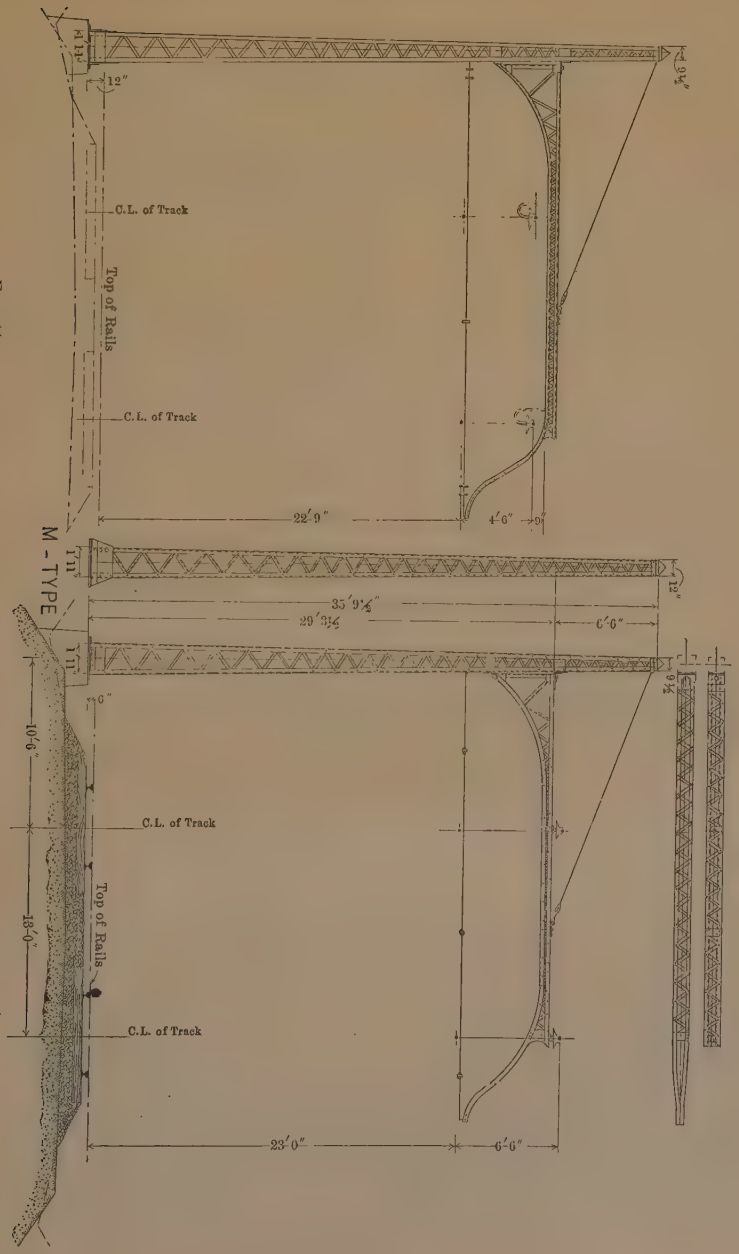


Fig. 11.

Fig. 12.

Mr. Coombs. As noted by Mr. Harte, some allowance for the weakening effect of brazing should be made in the assumed ultimate strength of trolley wire.

The limitation of wind in wooden poles can have little or no effect upon the cost of installation of the transmission line, since the specifications refer only to the crossings, the practical object of the requirement being to compel the use of selected sticks at these important points.

Overhead Construction for Electric Traction.—Both upright and suspended signals become more difficult of observation as the amount of overhead material, either longitudinal or transverse, is increased. Deep transverse trusses obscure the foreground of upright signals, and, with suspended signals, they form a background which moves vertically as the train approaches the signal. Double catenaries have a somewhat similar effect, and, in addition, compel a transverse shifting of signal posts from the position possible with single catenaries.

The writer cannot agree with Mr. Harte that the factors of safety of the supports, messengers, hangers, and trolley wire should be on a regularly descending scale. As a matter of fact, interference with operation will be due: first, to defects in the pantagraph; second, to weakness of the trolley wire; and finally, and at very rare intervals, to weakness in the messengers or their supports. Owing to the great inherent strength of the material of the messengers, the absence of the usual weakness due to shop work, and the fact that the first assumed condition of loading is of very rare occurrence, a factor of safety of 3 seems reasonable to the writer. However, a greater factor can readily be secured by an alteration of the sag or an increase in the gauge of the comparatively inexpensive steel wire.

No factor of safety for the hangers was stated, as the stresses in them are small, and their section will be governed by other considerations. Referring to the steel supports, any addition to the specified factor would probably be in the nature of a greater allowance of metal for corrosion, which is partly a detail of design and partly dependent upon the question of steam operation.

Mr. Mayer is incorrect in stating that there is no precedent for single catenary spans of 300 ft., as the Siemens-Schuckert construction described by the writer has been built with spans up to 328 ft., and the Syracuse, Lake Shore and Northern Railroad installation, described by Mr. Archbold, has spans of 300 ft.

Plate LXXI shows the counterweighting of the trolley wire in the Siemens-Schuckert construction, which, as surmised by Mr. Harte and the writer, does not necessitate the use of large pulley wheels.

The writer cannot agree with Mr. Mayer that very long pantagraph bows are necessary with the single catenary. They are most undesirable, from an operating standpoint, and presuppose lateral deflec-

FIG. 1.—GUARD BRIDGE, TO PROTECT RAILROAD TRACKS.

FIG. 2.—OVERHEAD CONSTRUCTION ON THE SEEBACH-WETTIGEN LINE.

FIG. 3.—OVERHEAD CONSTRUCTION ON THE BLANKENESE-HAMBURG-OHLSDORF LINE.

tions of the trolley wire, not borne out by practice, and more readily overcome by guying. Mr. Coombs.

With a view to decreasing the cost of construction and maintenance, it is desirable, and may be possible, to reduce the specified overhead clearance of 22 ft. above the rail. Except at grade crossings, it is not

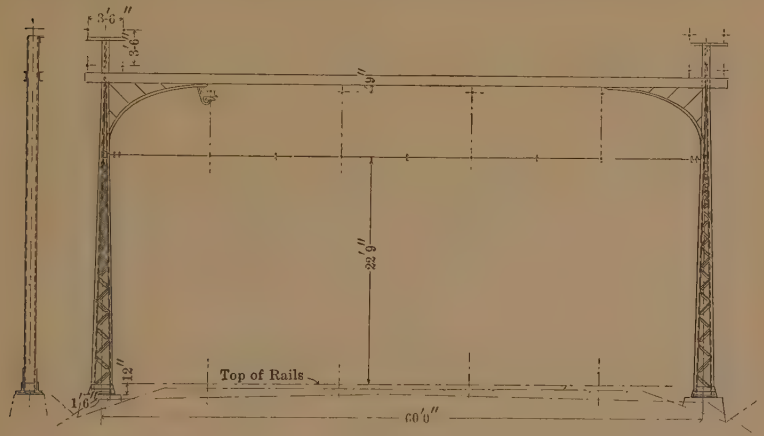


FIG. 13.

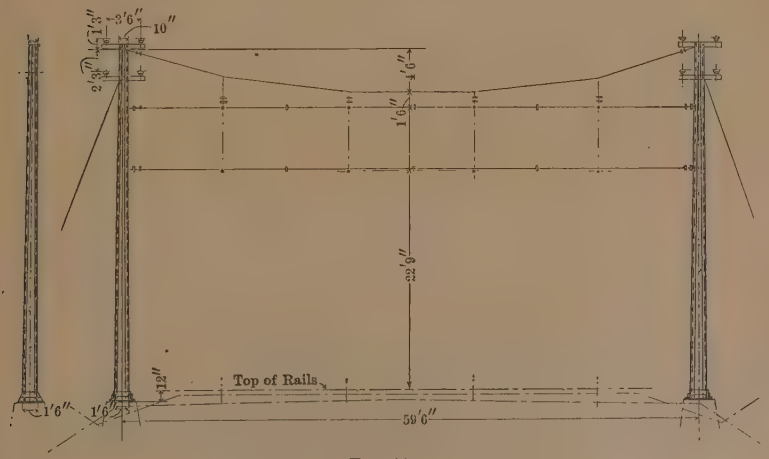


FIG. 14.

apparent to the writer that there is any advantage in providing a large clearance above the tops of cars. It would appear that such clearances, while desirable on steam roads, have no real usefulness on an electrified line. No feasible clearance would permit the operation of a track pile-driver under the wires, unless exceptional care were

Mr. Coombs. exercised in raising the leads and in handling material. On the rare occasions when pile-driving, or the operation of large cranes, becomes necessary, the adjoining superstructure may be temporarily guyed and the interfering catenaries removed.

Double-Track Brackets (300-ft. Single Catenary).—The specifications require a factor of safety of 3 with the infrequent combination of ice and wind loads, but a structure designed in this way will not resist successfully the forces developed by a broken messenger cable. Should a messenger break, two or more adjoining brackets would fail by bending, and more or less rotation of brackets would occur between the anchorages. Since the messenger cables have a factor of safety of 3 (which can be increased if desirable) under ice and wind loads, such failure might be regarded as dependent upon burning wrecks or other very unusual accidents. The writer is of the opinion that a suitable arrangement of guys, from the end of a bracket to the messenger cables, or to an auxiliary cable on the posts, can be designed, either to remove or curtail the effect of even broken messengers.

Otherwise, since the catenaries are underhung, their attachment may be made the weakest point, so that the accidental application of excessive forces would pull down one or more spans of catenary without injuring the remaining construction. Figs. 11 to 14 are submitted as proposed designs.

In conclusion, the writer believes that improvement in the design of pantagraphs, the use of trolley wire of greater inherent strength, with catenaries and supports providing uniform elasticity, will solve the important problems of overhead construction.

The Revised Specifications mentioned herein by the writer are as follows:

REVISED SPECIFICATIONS FOR OVERHEAD CONSTRUCTION OF HIGH-TENSION TRANSMISSION-LINE CROSSINGS.

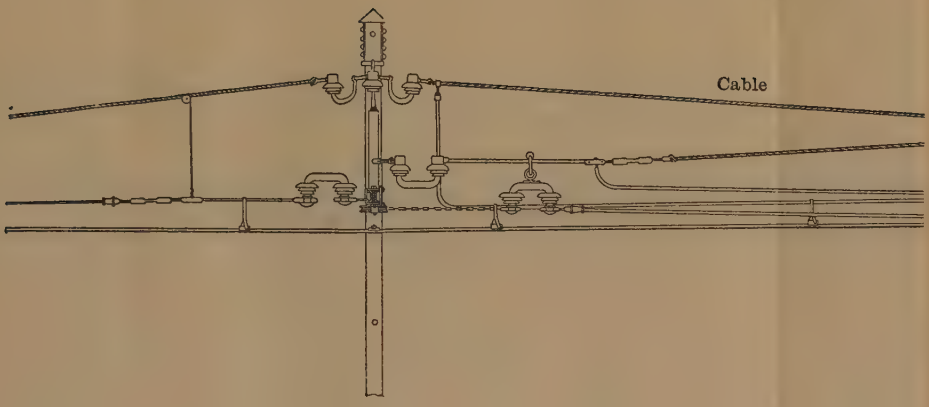
General Requirements.

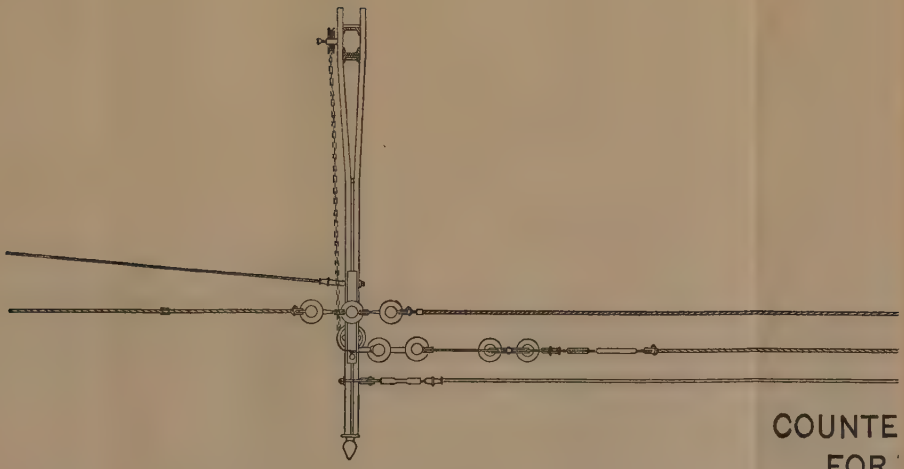
Drawings.—Complete drawings shall be furnished, in duplicate, for approval before construction is commenced.

General drawings shall contain full information covering stresses, span, normal sag, size and material of wires or cables, voltage, elevation of the points of support from the top of the rail, and the maximum sag and consequent clearance above the top of the rail.

Detailed plans shall show full details of insulators, pins, clamps, etc., and their supporting construction and foundations.

After approval, ... complete sets of prints shall be furnished for file, and, when the construction is at the expense of the ....., the original tracings shall be forwarded for file.





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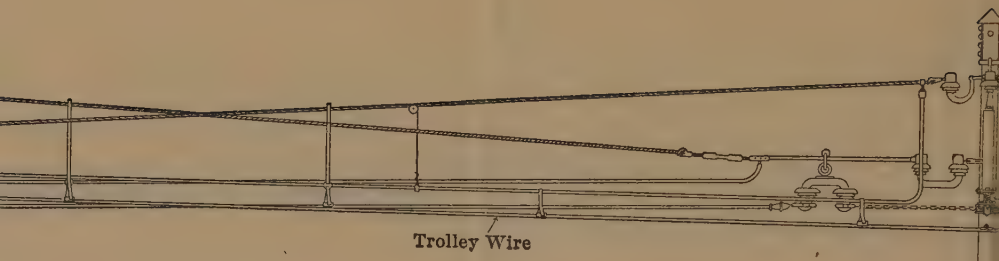
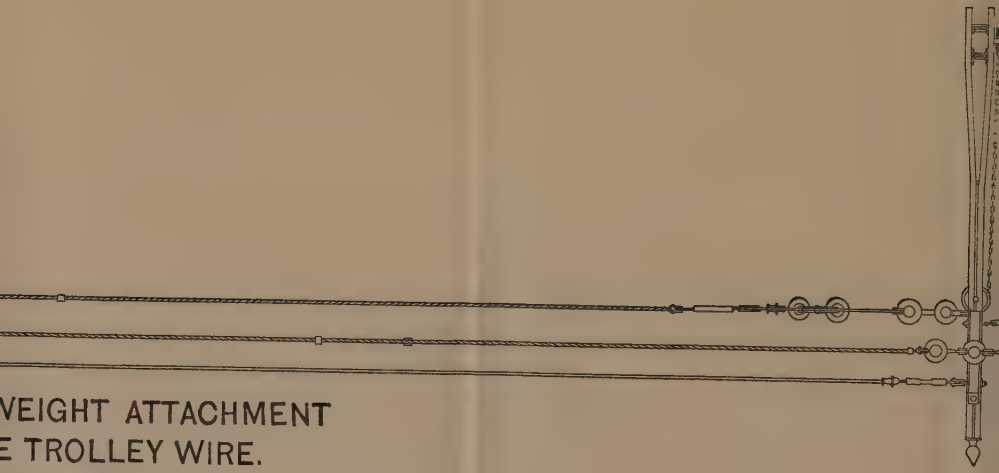
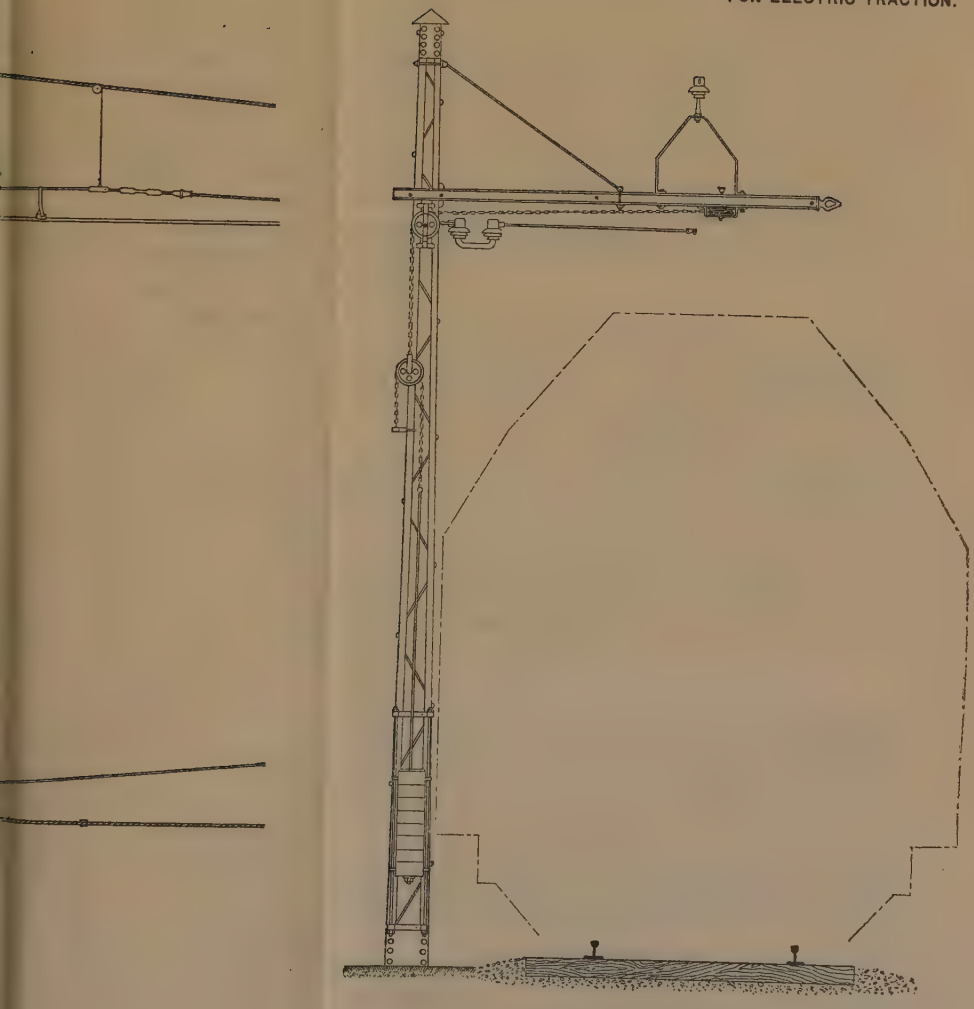




VEIGHT ATTACHMENT
E TROLLEY WIRE.
NS-SCHUCKERT TYPE.

PLATE LXXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LX, No. 1076.
COOMBS ON
OVERHEAD CONSTRUCTION
FOR ELECTRIC TRACTION.



Mr. Coombs.

Clearance.—The clear distance of any part of the construction from the center line of the roadway, and the clear head-room above the top of the rail, or the ground, shall be as specified by the proper official of the ....., but the clear head-room shall not be less than 35 ft. above the top of rail, nor less than 6 ft. above any existing wires, such clearance to obtain under the maximum deflection due to loads and temperature.

Spans.—The poles or towers supporting the crossing span and the two adjoining spans shall be self-supporting or be guyed so as to be in effect self-supporting. Anchor insulators, clamps, auxiliary attachments, etc., at the ends of the crossing span and the adjoining span shall be designed so as to prevent the wires from falling or pulling through, in case of failure in an adjoining span or at the support.

The poles or towers supporting the crossing span and the adjoining spans shall be in a straight line, and as nearly at right angles to the right of way as practicable.

Conditions of Loading.—

(1) ½ in. ice + 8 lb. per sq. ft. wind, on cylindrical surfaces.	On any two adjoining spans.
½ in. ice + 13 lb. per sq. ft. wind, on flat surfaces.	
(2) 14.5 lb. per sq. ft. wind, on cylindrical surfaces.	On any two adjoining spans.
24.0 lb. per sq. ft. wind, on flat surfaces.	
(3) ½ in. ice + 8 lb. per sq. ft. wind, on cylindrical surfaces.	On one span.
½ in. ice + 13 lb. per sq. ft. wind, on flat surfaces.	

and

½ in. ice + 2.0 lb. per sq. ft. wind, on cylindrical surfaces.	On the adjoining span.
½ in. ice + 3.3 lb. per sq. ft. wind, on flat surfaces.	

Loads.—

Dead load, or weight of the material;

Ice load: Weight of ice ½ in. thick all around exposed members.
Weight of ice = 0.033 lb. per cu. in.

Wind load:

8.0 lb. per sq. ft. of projected area of ice-covered wires (70 miles per hr.)

13.0 lb. per sq. ft. of projected area of ice-covered flat surfaces (70 miles per hr.)

2.0 lb. per sq. ft. of projected area of ice-covered wires (33 miles per hr.)

Mr. Coombs.

3.3 lb. per sq. ft. of projected area of ice-covered flat surfaces (33 miles per hr.)

14.5 lb. per sq. ft. of projected area of wires without ice (100 miles per hr.)

24.0 lb. per sq. ft. of projected area of flat surfaces without ice (100 miles per hr.)

Factors of Safety.—The factors of safety, under the combination of loads giving the maximum stress, shall be as follows:

Wires and cables	2½
Insulators, pins, clamps, steel cross-arms and connections	3
Steel superstructure	3
Wooden superstructure and cross-arms	5

Temperature.—In the determination of stresses and clearances, and in erection, provision shall be made for a change in temperature of 70° fahr. above and 80° fahr. below a normal temperature of 60° fahr.

Thickness of Material.—The material of the steel superstructure shall be not less than ¼ in. in thickness.

Radius of Gyration.—The length of any compression member shall not exceed 150 times its least radius of gyration.

Connections.—In general, the connections shall develop the full strength of the members, and shall be designed to avoid, as far as possible, induced stresses due to eccentricity.

Net Section.—In calculating tensile stresses, allowance shall be made for reduction in area, due to rivet holes (adding ⅛ in. to the nominal diameter), screw threads, etc.

Rivets.—Rivets shall be machine-driven, wherever practicable. Loose or defective rivets shall be carefully cut out and replaced; if necessary to avoid injuring the material, they shall be drilled out. The diameter of the finished rivet hole shall be not more than $\frac{1}{16}$ in. greater than the diameter of the cold rivet.

Bolts.—Bolts shall not be used in place of rivets, except as specified; and, when used, the holes shall be reamed and the bolts made to a close fit.

Straightening.—All material, when necessary, shall be carefully straightened at the shop before assembling.

Wires.—No line wire of a smaller size than No. 6, B. & S. gauge copper shall be used, or if of aluminum or other metal or alloy, than of a size equivalent in strength to No. 6 B. & S. gauge copper.

Guys.—Guy wires shall have an efficient anchorage, and be protected, or be of extra strength, at the ground level.

The material of guys shall be double-galvanized stranded steel, of not less than $\frac{5}{16}$ in. diameter.

Mr. Coombs.

Pins.—Insulator pins shall be of malleable iron, cast steel or other approved metals.

Insulators.—High-tension insulators of porcelain, or other approved material, shall be used throughout the crossing and adjoining spans, and shall be tested for not less than 30 000 volts, alternating current. Where more than 15 000 volts will be carried by the wires, the insulators shall be tested to double the voltage to be carried. Insulators or caps shall not be cemented during freezing temperatures.

One or more insulator units, as may be required, shall be assembled complete and mechanically tested to destruction.

Insulators shall be tested in accordance with special instructions to be given, and reasonable notice of the proposed tests shall be given the ....., and his representative shall be given every facility for witnessing them.

In all cases, full test reports shall be furnished.

Galvanizing.—Messenger and guy wires, insulator pins, clamps, etc., shall be galvanized. Structural-steel poles shall be galvanized in an approved manner, or shall be painted, as provided below.

One or more tests, as may be required, shall be made from each lot of material galvanized. The standard solution of sulphate of copper in water shall have a specific gravity of 1.185 at 70° fahr., and a temperature between 60 and 70° fahr.

Test pieces shall be given four immersions of 1 min. each, in the standard solution, and after each immersion shall be immediately washed in water and wiped dry.

If, after the fourth immersion, any copper color appears, or the zinc is removed from the test piece, the lot from which the test piece was taken shall be rejected.

Painting.—Structural steel shall be thoroughly cleaned at the shops and given one good coat of ..... All surfaces coming in contact in assembling shall be given one coat of ..... paint. Parts which will be inaccessible after erection shall receive two shop coats of..... paint. All machined surfaces shall be coated with white lead and tallow. After erection, all steel shall be given ..... coats of ..... paint. Painting will not be permitted during rainy weather, or when the surface of the metal is wet. All dirt, cinders, oil blisters, etc., shall be removed before painting.

Drainage.—Pockets, such as enclosed column footings, shall have drain holes, and shall be filled with water-proof material or concrete, or both, as may be required.

When the superstructure is embedded in a concrete foundation, the top of the foundation shall be sloped down toward the sides.

Foundations.—The foundations shall be designed to resist overturning, and shall extend above the ground as a protection to the lower part of the structure.

Mr. Coombs. Foundations, in general, shall be of concrete.

Timber.—Except as otherwise provided, wooden poles will not be permitted for the supports of the crossing span.

All timber shall be of the best quality of the kind and use specified, cut from sound trees, and sawed to size; it shall be close-grained and solid, and out of wind, free from defects, such as injurious ring shakes, crooked grain, large, unsound or loose knots, knots in groups, decay, large pitch pockets or other defects which would materially impair its strength.

Each wooden pole or tower shall be set in a concrete base, or the lower end shall be creosoted or otherwise treated with preservative.

Poles shall be of the first quality, with a minimum diameter of 8 in. at the top, and within the following limits of wind:

30 to 40 ft. long	not	more	than	3 in.
40 to 50 ft. "	"	"	"	4 "
50 to 60 ft. "	"	"	"	5 "
60 ft. and longer	"	"	"	6 "

Cross-arms, bearing surfaces, and the surfaces of notches, etc., shall be treated with paint or preservative.

Protection.—The poles or towers supporting the crossing span and the adjoining spans shall be grounded in an approved manner, and, if located on ..... property, shall be provided with small danger signs of an appropriate design.

Adjustment.—The sag given the wires at erection shall be adjusted to the temperature of the wires. Particular care shall be exercised in adjusting insulators, clamps, etc., to obtain the desired distribution of stresses.

Weight.—A variation, in section or weight of materials, of more than 2½% will be sufficient cause for rejection, except that sheared plates may vary according to the allowances of the manufacturers' standard.

Workmanship.—The workmanship on the various classes of construction involved shall conform to the requirements of first-class practice.

Unit Stresses.—The unit stresses in materials shall be as given in Table 9.

Composition of Steel and Iron.—The composition of the several kinds of steel, and of cast iron, shall be as given in Table 10.

Concrete.

The proportions of the materials in the concrete shall be as called for on the drawings, and the proportioning and method of mixture shall be as required by the Engineer.

*Cement.**—All cement used in the work shall be of an approved brand of Portland cement. The specific gravity of the cement, thoroughly dried at 100° cent., shall not be less than 3.10. It shall leave, by weight, a residue of not more than 8% on the No. 100 and not more than 25% on the No. 200 sieve. Mr. Coombs.

It shall develop initial set in not less than 30 min., but must develop hard set in not less than 1 hour nor more than 10 hours.

Tensile Strength.—The minimum requirements for tensile strength for briquettes 1 in. square in section shall be within the following limits, and shall show no retrogression in strength within the periods specified:

Neat Cement:

24 hours in moist air.....	150 to 200 lb.
7 days (1 day in moist air, 6 days in water)....	450 to 550 "
28 days (1 day in moist air, 27 days in water)...	550 to 650 "

One part cement, three parts sand:

7 days (1 day in moist air, 6 days in water)....	150 to 200 "
28 days (1 day in moist air, 27 days in water)...	200 to 300 "

Constancy of Volume.—Pats of neat cement, about 3 in. in diameter, ½ in. thick at the center, and tapering to a thin edge, shall be kept in moist air for a period of 24 hours.

(*a*)—A pat is then kept in air at normal temperature, and observed at intervals for at least 28 days.

(*b*)—Another pat is kept in water maintained as near 70° fahr. as practicable, and observed at intervals for at least 28 days.

(*c*)—A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely-closed vessel, for 5 hours.

These pats, to pass the requirements satisfactorily, shall remain firm and hard, and show no sign of distortion, checking, cracking, or disintegrating.

Sulphuric Acid and Magnesia.—The cement shall not contain more than 1.75% of anhydrous sulphuric acid (SO_3), nor more than 4% of magnesia (MgO).

Sand.—All sand shall be hard, clean, coarse, and sharp, and shall not contain more than 1.5% of clay or other foreign matter. If required by the Engineer, it shall be screened.

Stone.—All stone shall be sound, hard, and durable, and free from dirt and foreign matter, and shall pass through a ring 1½ in. in diameter.

Consistency.—The degree of moisture for mortar, grout, or con-

* Joint Committee on Concrete and Reinforced Concrete.

Mr. Coombs. crete shall be as required by the Engineer; in general, mortar shall be plastic, grout fluid, and concrete of such consistency that it will quake when being deposited.

Mortar, grout, or concrete which has commenced to set shall not be used in the work.

Placing.—Concrete shall be deposited in the work so that there will be no separation of mortar and stone. It shall be laid quickly in layers, and spaded as may be required. Rock surfaces shall be thoroughly cleaned, and earth surfaces shall be compacted in a satisfactory manner, before concrete is deposited against them. Surfaces of concrete against which fresh concrete is to be laid shall be cleaned and slushed over with grout, and shall be provided with a bond if required by the Engineer.

Forms.—Forms shall be of substantial construction, and designed to preserve the concrete in the form required by the drawings. All exposed surfaces of concrete shall be true to form and surface.

General Clauses.

1.—Every facility for the inspection of the materials and workmanship shall be furnished by the contractor; he shall furnish proper testing apparatus, and shall prepare and test such specimens as may be required.

2.—All work shall be subject to the inspection and approval of the ..... Engineer, and his interpretations of the drawings and specifications, and his decisions as to the quantity or quality of the work, shall be final and conclusive.

3.—The contractor shall remove all falsework, timber, or rubbish incident to his operations, and shall leave the site unobstructed and clean.

4.—The contractor shall bear the cost of any suit which may arise, and shall pay all damages which may be awarded in consequence of the use by said contractor of any patented device in the construction of any work under these specifications.

5.—The contractor shall obtain all necessary permits, and shall assume all risks of accidents to men or materials prior to the acceptance of the finished structure.

6.—The contractor shall not be entitled to payment for extra work or material unless such extra work or material has been specifically ordered, in writing, by the .....

MEMOIRS OF DECEASED MEMBERS

CHARLES PAINE, Past-President, Am. Soc. C. E.*

DIED JULY 4TH, 1906.

Charles Paine was born in Haverhill, New Hampshire, on April 25th, 1830, and died at Tenafly, New Jersey, on July 4th, 1906. Almost his entire working life, which began when he was 14 years old, was spent in railroad service, and the building of his professional position was in that period when the railroad art was still primitive and when the opportunities for education as a Civil Engineer, in the United States, were confined almost to actual work in the office and in the field.

Mr. Paine was descended from Stephen Paine, who came to the United States from England in 1638, and his family remained in New England for eight generations. He belonged to a fine, substantial stock, and some of his ancestors were distinguished.

Mr. Paine's school education was quite limited. In 1839, he entered the College of the Order of St. Sulpice, in Montreal, where he remained for two years and where he acquired that part of his school education which always seemed to him the most valuable, *viz.*, a good grounding in French and Latin and in the elements of what old-fashioned people call "polite education." He spent a year at an academy in Meriden, New Hampshire, and two years in New York at a school of which Mr. Charles Coudert was principal. This Mr. Coudert was the father of the late Charles and Frederic Coudert, eminent lawyers of New York and Paris. From an uncle, William T. Porter, editor and proprietor of the *New York Spirit of the Times*, young Paine, during these two years, was permitted certain opportunities to see life and people, and he always attached a good deal of value (and not without reason, perhaps) to the hours which he passed in the offices of that newspaper in the company of the wits, men about town, and famous actors and actresses. He says that in that brief period he "drank in a love of fine things in conduct, in art, in literature, and in manners which has continued a joy to me throughout my life."

In the spring of 1844, Paine's uncle, Governor Charles Paine, of Vermont, took the lad into the counting-room of his broadcloth factory, where he remained until August, 1845. Then he entered the service of the Vermont Central Railroad, the surveys for which had just been commenced. Of this enterprise, Governor Paine was President. Here he began as rodman in the corps of Calvin Brown, one of the old engineers of the period, who enjoyed a fine reputation.

* Memoir prepared by Edward P. North and H. G. Prout, Members, Am. Soc. C. E.

Other young men, afterward distinguished, who were associated with Paine in this work at this time, were the late S. M. Felton, M. Am. Soc. C. E., afterward President of the Philadelphia, Wilmington and Baltimore Railroad and of the Pennsylvania Steel Company; Mr. Charles Collins, afterward Chief Engineer of the Lake Shore and Michigan Southern; Mr. Carpenter, afterward United States Senator from Wisconsin; and Dr. Edward H. Williams, afterward General Superintendent of the Pennsylvania Division of the Pennsylvania Railroad, and a member of the firm of Burnham, Parry, Williams and Company, owners of the Baldwin Locomotive Works. Mr. Paine enjoyed the close friendship of all these men until their lives ended, and it is related that at the time of the Chicago fire, in 1871, Dr. Williams and his wife, knowing that the Paines lived in Chicago, immediately shipped by express from Philadelphia a complete outfit of clothes for each member of the Paine family, without stopping to ask if the clothes were needed.

In the autumn of 1847, the Vermont Central was completed into Northfield, and, for a very short time, young Paine got a chance to fire a locomotive, which he always regarded as one of the most valuable experiences of his life. The following winter he spent in the drafting rooms of Brown and Hastings, Civil Engineers, in Boston, and of Hinkley and Drury's Locomotive Works, where he made quite complete drawings of all the parts of a locomotive.

In 1848, Mr. Paine took charge of a division of the Vermont and Canada Railroad, under Henry R. Campbell, Chief Engineer. This road was completed in 1850. Mr. Paine then went to Montreal and took charge of the contracts of H. R. Campbell for building a railroad from Rouses Point to St. Johns, and for building a branch line from St. Lambert to intersect with the line of railway between St. Johns and La Prairie. At this time he also had charge of the building of docks at Moffatt's Island, opposite Montreal.

It will be seen that the young man had considerable responsibilities before he was of age, and he appears to have been in no way reluctant to assume still other responsibilities, for on May 13th, 1851, less than a month after reaching his majority, he was married to Olivia Blodgett Hebard, of Chelsea, Vermont. His wife belonged also to one of the most solid New England families. She was a woman of great cultivation of mind and of strong and beautiful character, and they lived together in the greatest happiness until Mrs. Paine's death in the summer of 1897. They had six children, and four sons now survive.

In 1855, Mr. Paine moved to Wisconsin, where he became Chief Engineer of the Beaver Dam and Baraboo Railroad, and of the Fox River Valley Railroad, neither of which enterprises got beyond the stage of grading the roadbed, because of the great panic of 1857. In

August, 1858, Mr. Paine became Superintendent of the Western Division of the Michigan Southern and Northern Indiana Railroad, which road was at that time five months behind in its pay-roll and physically pretty nearly a wreck. The local nickname for the road was the "Miserably Slow and Nearly Insolvent Railroad." These conditions, however, were not peculiar to that railroad in the year 1858. Mr. Paine's connection with this railroad and its lineal successors continued for twenty-three years. In January, 1864, he was made Chief Engineer of the road, and on March 1st, 1872, he became General Superintendent of the Lake Shore and Michigan Southern Railway. While in charge of this road he made such improvements and economies that by 1876 he had demonstrated his ability to carry freight for 4 mills per ton-mile, and from this, at the time, small sum, pay all the costs except for improvements, dividends, and interest on the bonded debt.

He remained Superintendent of the Lake Shore and Michigan Southern until he was appointed General Manager of the New York, West Shore and Buffalo, in August, 1881. He organized and carried through the building of this road, and upon its bankruptcy he found himself with health impaired and with the savings of his lifetime gone, for he himself had invested in the securities of the enterprise in which he believed enthusiastically.

In order to get himself in condition to rebuild his fortunes, he adopted the novel and bold scheme of traveling in Europe for a year on borrowed money. The remedy was characteristic and highly successful, and until the day of his death he never suffered another illness.

He served for a short time as the General Superintendent of the New York, Pennsylvania and Ohio Railroad, and for a few months as Second Vice-President of the Erie, and then he went to Pittsburg to help Mr. Westinghouse in developing the natural gas industry through the Philadelphia Company. There he remained until December, 1890, he having had active executive charge of the company.

He returned to New York at the end of 1890, and opened an office as Consulting Engineer, which office he maintained until 1899; part of which time, however, he was General Manager of the Union Steamboat Line, a subsidiary Erie company, and he occupied an important and confidential position in the administrative organization of the Erie.

From 1899 until a year before his death, Mr. Paine was General Manager of the Panama Railroad Company, and for a time he was also Vice-President and a Director of that Company. This service ended with the purchase of the Panama Railroad by the United States Government and the transfer of its management to the existing Canal Commission.

Mr. Paine was elected a Member of the American Society of Civil Engineers on its reorganization in December, 1867, and was the

second man to join the Society: numbering 17 on the list of members as it stood for the first year. He contributed to the *Transactions* Paper XX: "History of the Iron Rails on the Michigan Southern and Northern Indiana Railway," and was President of the Society during 1883.

At the time of his death, Mr. Paine was a Member of the Century Club, in New York, an Honorary Member of the Western Society of Engineers, and of the Engineers' Club of Cleveland, and a number of other scientific and philosophical bodies.

Mr. Paine's personality was so extraordinary, and meant so much to those among whom he lived, that special mention should be made of it. His manner was commanding, but singularly gracious. He had a dignified and impressive presence. He was of generous and enthusiastic temperament. He had a broad sympathy, wide reading, and a discriminating taste in literature and art; but, beyond this, there is much more to be said. In every generation there are a few men who impress their fellow men by beauty and nobility of character, quite apart from those qualities which we may think of as purely intellectual. They have a distinction which wealth or power or achievement cannot bestow. In the deepest recesses of our minds we recognize these men as being the real nobility—the flower of humanity. Mr. Paine belonged to the small group of men distinguished by character. He had intellectual superiority, and he was a man of honorable achievement; but we, who knew him well, think of him first and respect him most for the subtle qualities of gentle manliness. His temper was naturally quick, and he had great personal dignity; but his courtesy was unfailing and his modesty was sincere. He was chivalric in thought and conduct. Honor, truth, and duty were in the roots of his nature—inherited, bred in the bone. These were his shining characteristics, by virtue of which his life was lived in a high and serene atmosphere, and in that atmosphere dwelt with him a wife, his equal in every way.

CALVIN EASTON BRODHEAD, M. Am. Soc. C. E.*

DIED APRIL 29TH, 1907.

Calvin E. Brodhead was born in Pike County, Pennsylvania, on December 27th, 1846. His family moved to Mauch Chunk, Pennsylvania, in 1851. He attended school at what was known at the time as Park Seminary, and at St. Mark's Parish School, where Felix Ansart was Principal. During vacation periods, he worked in the blacksmith shop of N. Remmel and Company, who repaired cars for the old Beaver Meadow Railroad. When the survey was made for the railroad from Bethlehem to Bath, Pennsylvania, about 1862, he found employment on the engineer corps, and chose engineering as his profession. After the great flood of 1862 in the Lehigh River, which destroyed the canal above Mauch Chunk, he entered the service of the Lehigh Valley Railroad, locating the line over Wilkes-Barre Mountain, between Penn Haven and White Haven. On this work he met the late Sidney Dillon, F. Am. Soc. C. E., and a friendship was formed which lasted until Mr. Dillon's death.

After the Lehigh Valley Railroad was opened to Wilkes-Barre, Mr. Brodhead moved farther up the line to what was known as the Pennsylvania-New York Canal and Railroad. About 1871 he was transferred from Wilkes-Barre to Bethlehem, Pennsylvania, and, as Principal Assistant Engineer, under the late Robert H. Sayre, Chief Engineer, commenced locating the Easton and Amboy Railroad. He remained with the Lehigh Valley Railroad until 1877. During the building of this line (Easton and Amboy) the construction of the Musconetcong Tunnel was directly under Mr. Brodhead's charge.

From 1877 until 1883 he was engaged in the lumber business, and then he formed a partnership with Lafayette Lentz, of Mauch Chunk, John Byron and Daniel C. Hickey, of Mt. Vernon, New York, and engaged in the contracting business. The first contract of the new firm was for about ten miles of very heavy work on the Southern Pennsylvania Railway in Fulton County, Pennsylvania. In 1885 the firm secured the contract for the Vosburg Tunnel for the Lehigh Valley Railroad. In 1887 the firm of Brodhead and Hickey succeeded Lentz and Company, and, while connected with this firm, Mr. Brodhead was engaged on several large undertakings, notably the Palisade Tunnel for the New York, Susquehanna and Western Railroad, and a portion of the Pittsburg, Bessemer and Lake Erie Railroad, and the Lehigh Valley Railroad. After the death of Mr. Hickey, in 1894, the firm name was changed to C. E. Brodhead and Brother, and subsequently to the Brodhead Construction Company, under which name the firm continued work until Mr. Brodhead's death. It was in the contract

*Memoir prepared by F. H. Clement, M. Am. Soc. C. E.

business that Mr. Brodhead spent the most active part of his life, and in that he was best known and most successful. He was a man of quick ideas, and was a born locating engineer, in which capacity he was frequently called in consultation.

Mr. Brodhead continued to be interested for many years in the coal and lumber business, having large interests in Kentucky. He was twice married, and three children by his first wife survive him.

Mr. Brodhead was elected a Member of the American Society of Civil Engineers on February 21st, 1872.

NATHANIEL HENRY HUTTON, M. Am. Soc. C. E.*

DIED MAY 8TH, 1907.

Nathaniel Henry Hutton was born on November 16th, 1833, in Washington, D. C., and died in Baltimore, Maryland, on May 8th, 1907. His earliest ancestor in the United States of whom there is any record, John Strangeways Hutton, was born in New York City in 1684 and died in Philadelphia in 1792. His father was James Hutton, who married Salome Rich, of Boston, Mass., in Washington, D. C.

Following the example of his elder brother, the late William Rich Hutton, M. Am. Soc. C. E., "Harry" (as he was familiarly called by those who knew him well) entered the service of the United States at an early age, adopting the profession of civil engineering. Neither had the advantage of a collegiate education, but they did have the good fortune to grow up under the thorough training of those days, in the specially excellent schools of Alexandria and Washington, taught by men like Ben Hallowell, Abbot and others. They made good use of those early opportunities, and by industry, faithful attention to duty, and continual study of the theory of engineering and the works of able engineers, their own experience and unusual natural talents enabled them to pass through the lower grades of the profession with credit to themselves, and with the respect and ever-increasing confidence of their superiors in their integrity and high tone, until they had come to rank well among the engineers of their period in the special lines to which their attention was called.

Mr. Hutton's work as a surveyor and engineer, up to 1896, may be summarized briefly as follows:

He was U. S. Assistant Engineer on explorations and surveys for the Pacific Railroad west of the Missouri River, on the 32d and 35th Parallels, from 1853 to 1856, inclusive; Chief Engineer of the El Paso and Fort Yuma wagon road (Department of Interior) during 1857 and 1858; Surveyor on the western boundary of Minnesota (Department of Interior) during 1859 and 1860; U. S. Assistant Engineer on the defenses of Baltimore from 1861 to 1865; U. S. Assistant Engineer in charge of the improvement of the Patapsco River from 1867 to 1876, and on the Western division of the Virginia Central Water Line (survey 1874 to 1875); and from 1876 until his death he was Engineer to the Harbor Board of Baltimore; he was also U. S. Assistant Engineer in charge of surveys for a ship canal to connect the Chesapeake and Delaware Bays during 1878 and 1879; Consulting Engineer for a project for a ship canal between Philadelphia and the Atlantic Ocean in 1894 and 1895; and Consulting Engineer for a projected ship canal to connect Lake Erie and the Ohio River in 1895 and 1896.

* Memoir prepared by William P. Craighill, Past-President, Am. Soc. C. E.

For many years previous to 1896, and up to the time of his death, Mr. Hutton had been Chief Engineer to the Harbor Board of the City of Baltimore. That he held this office so many years, during the administrations of mayors and councils of opposing political parties, is proof that his services were considered so valuable as to be almost indispensable. Later, he became President of the Harbor Board, as well as Chief Engineer.

The following tribute from the Harbor Board shows the high esteem in which he was held by his associates, and it may be said with truth that this was the sentiment of the business men of Baltimore who were best acquainted with his work and ability:

"The death of Major Nathaniel H. Hutton, Engineer of the Harbor Board of Baltimore City, comes at a time and under conditions which cause especially deep feelings of sorrow and regret in the minds of the members of the Harbor Board.

"Immediately after the fire of February 7th and 8th, 1904, he was called upon by the citizens of Baltimore to suggest and design plans for the new docks and the improvements of the harbor of this City. The preparations of these plans, together with his other duties as engineer of the Harbor Board, devolved upon him a very great amount of skillful professional work, and it is probable that he unconsciously overtaxed his strength in this way.

"The influence which Major Hutton has exerted upon the plans for the improvement of the Harbor, cannot be estimated. He has not lived to see the realization of what he has planned, but there can be no doubt that his activity and experience in this great work will be appreciated by his successors, and the citizens of Baltimore, when the full effects of his labors and efforts are realized.

"Major Hutton was an engineer of rare ability and of vast and varied experience. He was a gentleman of the old school, and a most faithful engineer and honest public servant.

"*Resolved*, that in the death of Major Nathaniel H. Hutton, the City of Baltimore has been deprived of a noble and trusted citizen and a capable and conscientious public servant, who has devoted many years of his life to her interests.

"*Resolved*, that the members of the Harbor Board, who particularly appreciate the full measure of loss suffered by his death, tender their sympathies to the family of the deceased, and that these Resolutions be spread upon the Minutes of the Board."

There are also appended resolutions adopted May 19th, 1907, by the Board of Public Improvements, of which Mr. Hutton was a prominent member:

"At a special meeting of the Board of Public Improvements held this date called to take action on the death of Major N. H. Hutton, President and Chief Engineer of the Harbor Board, the following resolutions were adopted:

"*Resolved*, that by the death of Major Hutton the City of Baltimore has lost a most faithful and efficient public officer, whose long service

as Harbor Engineer here and extended experience on important public works elsewhere made his services invaluable to this city.

"Also by his death, we, his fellow members of the Board of Public Improvements, have lost a trusted friend and wise counsellor, whose uniformly genial and courteous nature greatly endeared him to us.

"We extend to his family our sincere and heartfelt sympathy in their great sorrow."

Mr. Hutton was a Charter Member and Vice-President of the Engineers' Club of Baltimore. At his death the Club took the following action in his honor:

"*Whereas*, We, the members of the Engineers' Club of Baltimore, have learned with sincere sorrow of the death of our fellow member, Major N. H. Hutton; and whereas we recognize his earnest efforts, as a Charter Member and Vice-President, to promote the welfare of the Club, and the active, friendly and generous interest, manifested by him, in establishing its success:

"*Resolved*, that in his death the Engineers Club of Baltimore has been deprived of a distinguished member and a Loyal and Honoured Friend."

Mr. Hutton was also an architect of decided ability, as is shown by the outcome of the designs proposed by the firm of Hutton and Murdock, of which he was a member for several years, for the construction and alteration of a number of churches, dwelling-houses and warehouses in Baltimore, Washington, Virginia and Pennsylvania. One of his designs for a highway bridge in Baltimore was considered by a very judicious board to be the best among five that were submitted. Not only was Mr. Hutton esteemed as an able engineer and architect and a capable and faithful official, but he was admired and loved by his friends in an unusual degree. A few extracts are appended from many testimonials that have been received as proof of the statements already made.

After a long intercourse, under conditions which often test men's character, long-drawn-out surveys among the rough surroundings of camp life, in the midst of Indians and uncultivated and often lawless frontier people, both male and female, one of his closest friends writes:

"'Tis said that you must sleep with a man to learn his peculiarities. Well, if this is true, Harry and I ought to have become pretty well acquainted, for the nights we stretched ourselves on the ground under the same blanket, ate our grub out of the same tin pan, and drank our coffee out of the same tin cup, ran through years, and during the entire time our affection became closer. It was only necessary to know him to love him, and, of the many acquaintances I have made during a long and varied life, I have yet to meet the man who excelled him in the noble qualities of head and heart which he possessed. He was one of Nature's noblemen, a conscientious Christian whose only fear, if he knew what fear was, was to do wrong, and whose sense of honor was as firmly fixed as the everlasting hills."

Another, with whom Mr. Hutton had close professional and personal contact in Baltimore, gives the following high testimonial from himself and others of their mutual associate:

"All of us had the highest appreciation of his ability as an engineer and of the value of his services to the city. He had been our Harbor Engineer for so many years that he had become indispensable in the working of our city government. His advice was frequently sought by municipal engineers and other municipal officials, and his opinion was always respected on all engineering questions. He was progressive, broad and liberal in his views, yet conservative enough to hold down some of us younger and rasher engineers. He was a conciliating and harmonizing influence at all gatherings of engineers and meetings of boards and commissions. His personality was such, and his manner was so genial and kindly, that he could regulate or harmonize where others could not, and yet always retain the regard and affection of his associates.

"Because of his years of experience and of his broad learning, his place in our municipal government will be hard to fill. His place in our affections can never be filled."

Another who had served with Mr. Hutton very closely for many years adds:

"As an engineer, he was capable, careful, eminent and prominent, and was consulted in the development of many projects of National importance. On undertaking any new work he sought the results and opinions of others of distinction and after giving careful consideration formulated his plans.

"As a public official, he was earnest, honest and faithful, possessing a keen power of penetration, and his approval always carried weight.

"As a man, he was modest and retiring, affable and lovable, with ever a kind word for his fellow-man, be he high or low, and all in all a splendid type of a gentleman."

Still another says:

"I was thrown in intimate relations with him. He was always to me the embodiment of a true gentleman, in the highest and best sense of that word; honorable and truthful, above suspicion, always courteous and always manly.

"As an engineer, he was well trained and on broad lines. I had great confidence in him, and frequently consulted him about difficult problems coming up in my work, and always got sound and helpful advice. If I were called upon to name some special characteristic of Major Hutton, which distinguished him as an engineer, I should say that good judgment was his strong point.

"His death leaves a great blank, both professionally and socially. My feelings for Major Hutton were those of real, genuine affection, and I believe that most men who came in close contact with him had the same. It is difficult to imagine a true man having any sentiments for Major Hutton other than those of the profoundest confidence and respect."

The writer knew Mr. Hutton for more than forty years, both professionally and socially, and can fully bear testimony to the fact that what is said by others in what precedes is not exaggerated. His domestic life was charming and lovely.

In early manhood, Mr. Hutton married Miss Meta Van Ness, daughter of Colonel Eugene Van Ness, of the United States Army, who was a member of the well-known and distinguished family of that name in the State of New York. One of Mrs. Hutton's ancestors was Admiral Van Ness, of Holland, who lived in 1653; and in Scotland her lineage dates distinctly and honorably at least to 1542.

Mr. Hutton passed from time to eternity in May, 1907, and his devoted wife followed in September. They left three children, all resident in Baltimore, Mr. Harry Hutton, Mrs. S. S. Busby and Mrs. C. H. Wyatt.

Mr. Hutton was elected a Member of the American Society of Civil Engineers on June 3d, 1896.

GEORGE THOMAS NELLES, M. Am. Soc. C. E.*

DIED NOVEMBER 15TH, 1907.

George Thomas Nelles, son of George W. Nelles and Virginia Hobbs Nelles, was born on April 15th, 1856, in Muscatine, Iowa.

His boyhood was spent in Leavenworth, Kansas, to which place his parents moved in the summer of 1857. Mr. Nelles prepared for college in the private school of the Reverend (now Bishop) John Mills Kendrick, and was graduated from the Rensselaer Polytechnic Institute with the degree of C. E. in June, 1877.

After a few months' work as instrumentman with the United States Engineer Corps at Leavenworth, he entered the service of the Kansas City, St. Joseph and Council Bluffs Railway, as Assistant Engineer in charge of surveys and relocation.

In the summer of 1878 he re-entered the Government service, as United States Assistant Engineer, on the Missouri River improvement, having in charge at various periods the work at Atchison, St. Joseph, and Leavenworth, until the spring of 1883 when he was elected City Engineer of Leavenworth, Kansas. Entering upon his duties at a time when the city was growing rapidly, he directed much public work, supervising, during his term of office, the expenditure of more than $1 250 000 in grading and paving streets and constructing sewers, culverts, and bridges.

During his term of six years as City Engineer, Mr. Nelles was also Consulting Engineer for the Western Home for Disabled Volunteer Soldiers; Chief Engineer of the Riverside Coal Company; Chief Engineer of the Leavenworth Rapid Transit Company; and Chief Engineer of the East Omaha Land Company.

At the organization of the Nebraska and Colorado Stone Company, in 1889, Mr. Nelles became its Secretary and Manager. The company operated quarries in Nebraska and Colorado, contracting not only to furnish stone, but, also, in many cases, for the complete erection of the structure.

Severing his connection with the Stone Company in 1891, he entered the general contracting business, constructing sewers, pavements, bridges, water-works, and river and harbor improvements. The largest and most important contracts handled during the four years he spent in this work were the construction of the sewers in Denver, Colorado, and the harbor improvements in the Mississippi River at St. Louis, Missouri.

In the spring of 1895 Mr. Nelles again entered the Government service as U. S. Assistant Engineer, on the Tennessee River improvement at Chattanooga, Tennessee. During his six years of service on the Tennessee River and its tributaries, many important and difficult

*Memoir prepared by F. E. Bissell, M. Am. Soc. C. E.

problems presented themselves. He made a careful study of the construction of locks and dams under the conditions of fluctuating velocity of current and volume of discharge which there prevail. His reports on all subjects assigned to him were always exceedingly full and complete. He prepared detailed tables showing the cost of construction of the lift and guard locks at Colbert Shoals, Alabama. He investigated the discharge of the Tennessee River, checking the formulas with the actual measured velocities, and determining for this stream the value of n, in Kutter's formula. His solution of the problem of the effect of a dam on a submerged discharge, and on the surface level of the upper pool, reached in his study of projects for the improvement of that part of the Tennessee River known as the "Suck," is a material addition to engineering knowledge.

Mr. Nelles studied the conditions on the French Broad River, and made plans for widening and deepening the channel; he examined and reported on the necessity of making any improvement of Powells River; made plans and estimates for the low-water improvement of the Hiawassee, Little Tennessee, and Clinch Rivers, and also reported on the feasibility of making improvements on the Holston River.

Careful attention to details, and a comprehensive consideration of all the component parts of the subject, are features of each of these reports. They show that rare combination, complete theoretical knowledge and practical ability.

In June, 1901, Mr. Nelles was transferred to Cleveland, Ohio, as U. S. Assistant Engineer in charge of the improvements of the harbors on Lake Erie at Cleveland, Lorain, and Fairport. The same thoroughness and attention to details, combined with indomitable energy and great administrative ability, characterized his work there.

The earnestness with which he worked, the ability which he brought to the work, and the honesty of his dealings, combined with his cheerful disposition, made him a very companionable man, both socially and professionally.

His health began to decline in 1903. Two surgical operations failed to give more than temporary relief, and he died at Cleveland, Ohio, on November 15th, 1907.

On February 15th, 1884, Mr. Nelles was married to Miss Lena Ralston, who, with one son, survives him.

Mr. Nelles was a Member and a Director of the Civil Engineers' Club of Cleveland. He was elected a Member of the American Society of Civil Engineers on October 3d, 1888, and contributed to the *Transactions* a discussion* on the paper by the late George W. Rafter, M. Am. Soc. C. E., entitled "On the Flow of Water over Dams;" and also a discussion† on the paper by the late R. C. McCalla, M. Am. Soc. C. E., entitled "Improvement of the Black Warrior, Warrior, and Tombigbee Rivers, in Alabama."

* *Transactions*, Am. Soc. C. E., Vol. XLIV, p. 359.

† *Transactions*, Am. Soc. C. E., Vol. XLIX, p. 284.

HERBERT FRANKLIN NORTHRUP, M. Am. Soc. C. E.*

DIED JANUARY 21ST, 1908.

Herbert Franklin Northrup, born on a farm near Shoreham, Vermont, on October 9th, 1850, was the youngest child of Nazro and Mary Hawes Northrup.

After attending the village school he prepared for college in Kimball Union Academy, and entered Middlebury College, Vermont, in the class of '73. He next taught mathematics and English for two years at a boys' school in Flushing, Long Island. He then took a graduate course in engineering, in Sheffield Scientific School, Yale, in the class of '78.

His first engineering engagement was upon the Lake Champlain breakwater at Swanton, Vermont, in 1878, and in 1879 he was engaged on railroad maintenance work at Salem, Massachusetts. In the spring of 1880 he entered the employ of the Texas Pacific Railroad, as Assistant Engineer of construction, and was located at Fort Worth, Texas. He continued in the employ of that company, in responsible positions, until the completion of its construction in 1885.

On February 2d, 1882, he was married to Miss Cornelia F. Allan, of New Haven, Connecticut.

He entered the employ of the Missouri Pacific Railroad Company in 1885, as Assistant Engineer of construction in Missouri and Kansas, and in September, 1886, he engaged with W. V. McCracken and Company, Contractors, as Chief Engineer on the construction of railroads in Ohio and Indiana. From August to November, 1887, he was engaged as Locating Engineer on the Duluth, South Shore and Atlantic Railway, in the northern peninsula of Michigan.

In November, 1887, the writer engaged him as engineer in charge of preliminary and location surveys for the Chicago and West Michigan Railway, from Baldwin to Traverse City, Michigan, 75 miles, which was finished in June, 1888. He was then engaged until June, 1889, upon some construction work in the East, when he again returned to take charge of the construction of the road from Baldwin to Traverse City, following which he had charge of the location and construction of an extension of about 90 miles from Traverse City to Petoskey, Michigan, which was completed in 1893. In 1893 and 1894 he was engaged with the Detroit, Bay City and Alpena Railroad, and from 1895 to 1901 was in private practice and City Engineer of Traverse City, Michigan, and designed a proposed water supply for that city. During this time he also located several miles of road for the Lake Superior and Ishpeming Railroad Company.

In 1902 he entered the employ of the Cleveland, Cincinnati,

* Memoir prepared by J. J. McVean, M. Am. Soc. C. E.

Chicago and St. Louis Railroad Company, in charge of a residency on the relocation and construction of its line for double track, and reduction of grades and curvature, where he had charge of some very difficult and heavy work, especially the construction of several large-span concrete arches.

In May, 1905, he formed a partnership with the writer, as Consulting Engineers, with office at Grand Rapids, Michigan, where he was engaged until his death.

Mr. Northrup was beloved by all who knew him. He was of a very modest and retiring disposition, amiable, a staunch friend, and a thoroughly honorable business man. Quiet and even-tempered, honest in all his dealings, he had not only the entire confidence of his employers, but also the love and friendship of his assistants.

One of his many assistants, now occupying a responsible position with the City of Buffalo, says: "I knew him as a gentleman and an engineer, and nothing can be added to that. His even temper and kindly ways always left a pleasant recollection."

His death was very sudden and unexpected; after a severe fall on an icy sidewalk, he was attacked with prostatitis, necessitating an operation from which he did not rally.

Mr. Northrup was a Royal Arch Mason, a member of the Delta Kappa Epsilon College Fraternity, and was elected a Member of the American Society of Civil Engineers on January 6th, 1892.

JOHN LARKIN THORNDIKE, M. Am. Soc. C. E.*

DIED OCTOBER 12TH, 1901.

John Larkin Thorndike was born in Malone, Franklin County, New York, in 1834, and died in Lima, Peru, on October 12th, 1901.

In 1852 he began his professional career with an engineering party on the Ogdensburg and Lake Champlain Railroad, now a part of the Vermont Central System.

After this he went to the London and Port Stanley Railroad, in Upper Canada, as an Assistant Engineer, and later went still farther west and took a similar position on the Detroit and Milwaukee Railroad, in Michigan. Before this road was completed, Mr. Thorndike made a contract with the late A. W. W. Evans, M. Am. Soc. C. E., to go to Chili, South America, as an Assistant Engineer on the Copiapo and Chanarcillo Railroad, and in 1857 he began the work that made his reputation. Soon afterward he was made Resident Engineer on the Chanarcillo and San Antonio Railroad, and still later was given a similar position on the branch to Tongoy. His responsibilities increased with his reputation, and he was next put in charge of a very difficult portion of the road from Valparaiso to Santiago. When that was finished he went to the division between Santiago and Tiltil on the Ferro Carril del Sur, the trunk line south from the capital of Chili, now the most important line in that country. The work was very heavy and difficult. One part, which caused Mr. Thorndike much trouble and anxiety, was the masonry for the bridge over the Mapoche River; and in after years he took much satisfaction in seeing how well it had resisted the force of that torrential mountain stream.

Early in 1868, the Peruvian Government made a contract with Mr. Henry Meiggs, who had become famous as a builder of railroads in Chili, to build a road from Mejia to Arequipa, and he put Mr. Thorndike in charge of it, as Chief Engineer. The road is now 108 miles long, and runs from Mollendo to Arequipa. Mollendo, as a port, is a little better than Mejia, but it is only an open roadstead which cannot be used in rough weather.

Arequipa, the metropolis of southern Peru, is an inland city more than 7 800 ft. above sea level. Nearly half the road is in a desert, without water or vegetation. Men were rushed in, a few days after Mr. Thorndike arrived, and, before any instrumental surveys could be made, a large force of Chilean laborers were landed. He made a tangent across the flat part of the desert with only his field glasses, and put them to work. When that part of the line was staked out, it was found to need but little changing. The western end of the

* Memoir prepared by James R. Maxwell, M. Am. Soc. C. E.

tangent is on a plateau about 2 000 ft. above Mejia, and the development he made there, "The Cahuentala," is a model as a piece of location. It is 15 miles long, 5 miles in an air-line. There was no fresh water on the road except at the upper end, and the problem of supplying the graders with this necessity was very serious and expensive. The road and the Town of Mollendo are now supplied by a pipe line about 80 miles long, the ends of which differ in elevation about 6 000 ft.

Before the road was finished Mr. Meiggs made another contract with the Peruvian Government to build a number of roads, and Mr. Thorndike was made Engineer in Chief of all these roads south of Lima. These were the Glo and Moquega, 70 miles long; the Arequipa and Puno, 230 miles, and the Juliaca and Cuzco, 210 miles. The two latter, with the Arequipa and Mollendo, form the Southern System of Peru. The first, from Glo to Moquega, is a local road from the seaport to the interior. The second of these begins at Arequipa at an elevation of about 7 800 ft., and crosses the western range of the Andes through the Las Cruces Pass at an altitude of 14 630 ft., and then descends to Puno on Lake Titicaca, which is about 12 000 ft. above the sea. This road was finished in 1874.

The Juliaca and Cuzco leaves the Puno Railroad at Juliaca, 30 miles from its terminus, and runs almost due north. It crosses the main range at a slightly lower pass than that of the other road, and descends along a branch of a tributary of the Amazon. The lowest part of the road is 10 050 ft. above tide water. The grading was started at both ends, and also on both slopes from the summit. In 1879 the track reached Santa Rosa, one-third of the way to Cuzco, when the work was stopped on account of difficulties between the Government of Peru and the contractor. No more construction work was done on railroads for a number of years, and during that time Mr. Thorndike had charge of the operation of the Southern System.

In January, 1890, he made a contract to finish the Oroya (now the Central of Peru) Railroad in three years. At that time the track was laid to Chicla, a town high up on the western slope of the Andes, 87 miles from the sea and 12 215 ft. above it. Oroya, on the other side of the mountains, was 50 miles away, and at about the same elevation. All the grading on the last 30 miles had been done, but there was some very heavy work between, including fifteen tunnels and several large bridges. The summit (in the last tunnel) is 15 666 ft. in altitude. All the larger bridges were erected on the ground, which was a trying and tedious task, as the altitude was more than 15 000 ft. All the work was above 12 000 ft., and many difficulties were encountered, that would not be found at a lower altitude, but the road was built at the time specified—an example of Mr. Thorndike's remarkable executive ability.

This was the last important work of this modest, unassuming, and laborious man.

"His last years were devoted mostly to rest, but were also filled with small works, which showed his constant desire to be useful to his fellow man."

Mr. Thorndike was a magnanimous and lovable man, and all who knew him were his friends.

He was elected a Member of the American Society of Civil Engineers on May 7th, 1873.

WILLIAM ROBERTS, Assoc. Am. Soc. C. E.*

DIED DECEMBER 28TH, 1907.

William Roberts was born in Watertown, Massachusetts, on March 25th, 1835. He was a son of John Roberts, a descendant of one of the old Boston families. His parents moved to Waltham soon after his birth, and he attended the Waltham Public Schools, the private school of Daniel French, and the Allen School of Newton.

He entered the employ of the Boston Manufacturing Company, in the machine shop, as a start toward the development of his mechanical genius. Obtaining permission from the Fitchburg Railroad, he ran an engine from Waltham to Boston a number of times. He then went to Virginia, where he studied at the establishment of the Norfolk Manufacturing Company. When very young he entered the United States Navy. He was Third Assistant Engineer under Commodore Perry when he opened the Ports of Simoda and Hakodadi, in Japan, and served on the *Michigan,* on the Great Lakes in 1856, and on the steam frigate *Roanoke,* on the Coast of Central America in 1857. He was one of the officers on the steamer *Fulton* which captured Walker, the filibuster, in 1858, and he served on the *Memphis* on the Paraguay expedition in 1859.

In July, 1858, Mr. Roberts was promoted, becoming Second Assistant Engineer, and one year later he was made First Assistant. He resigned in September, 1859, but, in response to his country's call, reenlisted in the Navy in April, 1861. In 1863 he became Chief Engineer.

During the attacks on the forts and batteries at Pensacola Bay, in 1861, he was on the frigate *Niagara;* the steam sloop *Housatonic* carried him to a point off Charleston, in 1862, when she drove two iron-clad rams into port. He was attached to the frigate *Niagara* repairing at Charlestown Navy Yard, during 1863 and 1864.

After his retirement from the Navy he returned to Roberts' Crossing, Waltham, and joined his father in the manufacture of paper, and, even after his father's death, he carried on the business under the firm name, John Roberts and Son.

The manufacture of roofing paper was the principal product of the mill until his ever-active mind turned to the then new article, asbestos, and his mill was the first to produce asbestos fire-proof paper, the secret of the process being held by him for many years.

He declined the acceptance of public office, notwithstanding the many entreaties on the part of his friends. The only State positions he held were Commissioner on Prisons, and Representative to the

*Memoir prepared by Sumner Milton, Esq.

General Court. He was a staunch Republican, and was sent as a delegate to the State Convention for many years. He was a Member of the Waltham Board of Cemetery Commissioners, and a Director of the Waltham National Bank.

Mr. Roberts was a life member of Monitor Lodge, A. F. and A. M., and Post 29, G. A. R., of Waltham. He belonged to the Military Order of the Loyal Legion of the United States, also the American Society of Mechanical Engineers.

"He serves God well, who serves his creatures," truly speaks the life of William Roberts. Never was he known to refuse to help a worthy person or project. Many leave public bequests and are thought generous, but Mr. Roberts' method was to give continuously; and, as was his nature, quiet, just, liberal, honest, and philanthropic, so was his giving, and there are many individuals and institutions who miss his beneficence.

Mr. Roberts was an interesting conversationalist, having toured the world. He was especially interested in the ocean, and crossed the Atlantic on all the finest new steamers, his knowledge of mechanical engineering enabling him to note all the latest improvements in the engines. It was difficult, indeed, to ask a question on country or product on which Mr. Roberts could not give valuable information, and in such a simple manner that a child could enjoy his talk.

On October 27th, 1879, Mr. Roberts married Eva C., daughter of Hon. Gideon Haynes, and their home was always at Waltham. His married life was one of devotion, and it would be difficult to decide whether the palm should be given to him or to his companion in life.

William Roberts was elected an Associate of the American Society of Civil Engineers on June 4th, 1884.

PAUL ERNEST OBERNDORF, Jun. Am. Soc. C. E.*

DIED OCTOBER 13TH, 1907.

Paul Ernest Oberndorf was born at Centralia, Kansas, on February 16th, 1885, and died of typhoid fever at the same place after a short illness, on October 13th, 1907.

His childhood and early youth were spent on the farm. In the autumn of 1900, he entered the Friends Preparatory School at Washington, D. C., and was graduated with honors in 1902. He entered Princeton University in September, 1902, and was graduated four years later with the degree of Civil Engineer. His record in college was of such character that, upon his graduation, he was offered an assistant professorship, but he chose rather to enter at once into the practical work and experience of his profession.

He was first employed by Messrs. Waddell and Hedrick, of Kansas City, Missouri, as an assistant engineer upon the large viaduct, then building, between Kansas City, Missouri, and Kansas City, Kansas. After a short time he was sent as an assistant engineer on the substructure for a bridge over the Atchafalaya River, in Louisiana. This position required a man of judgment and precision, and Mr. Oberndorf did his work in such a manner as to mark him as a man of rare tact and ability for one so young. For the few months before his death, he was employed as draftsman in the office of Ira G. Hedrick, M. Am. Soc. C. E., and here, as in the field, his work showed him to be original and resourceful. Judging from the results of the little more than a year's work between his graduation and his death, Mr. Oberndorf undoubtedly had the qualities of a great engineer.

Mr. Oberndorf's temperament was a most happy one, his manner was cordial and friendly, and showed regard for the opinions and feelings of others. It is rare that one finds a more symmetrically developed, well rounded out man, in the highest sense of the term. His death was a great personal loss to his host of friends, for all who knew him loved him and were drawn to him by his gentleness, his manliness, and his magnetic personality.

Mr. Oberndorf was elected a Junior of the American Society of Civil Engineers on February 5th, 1907.

* Memoir prepared by L. R. Ash, Assoc. M. Am. Soc. C. E.

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OF THE

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